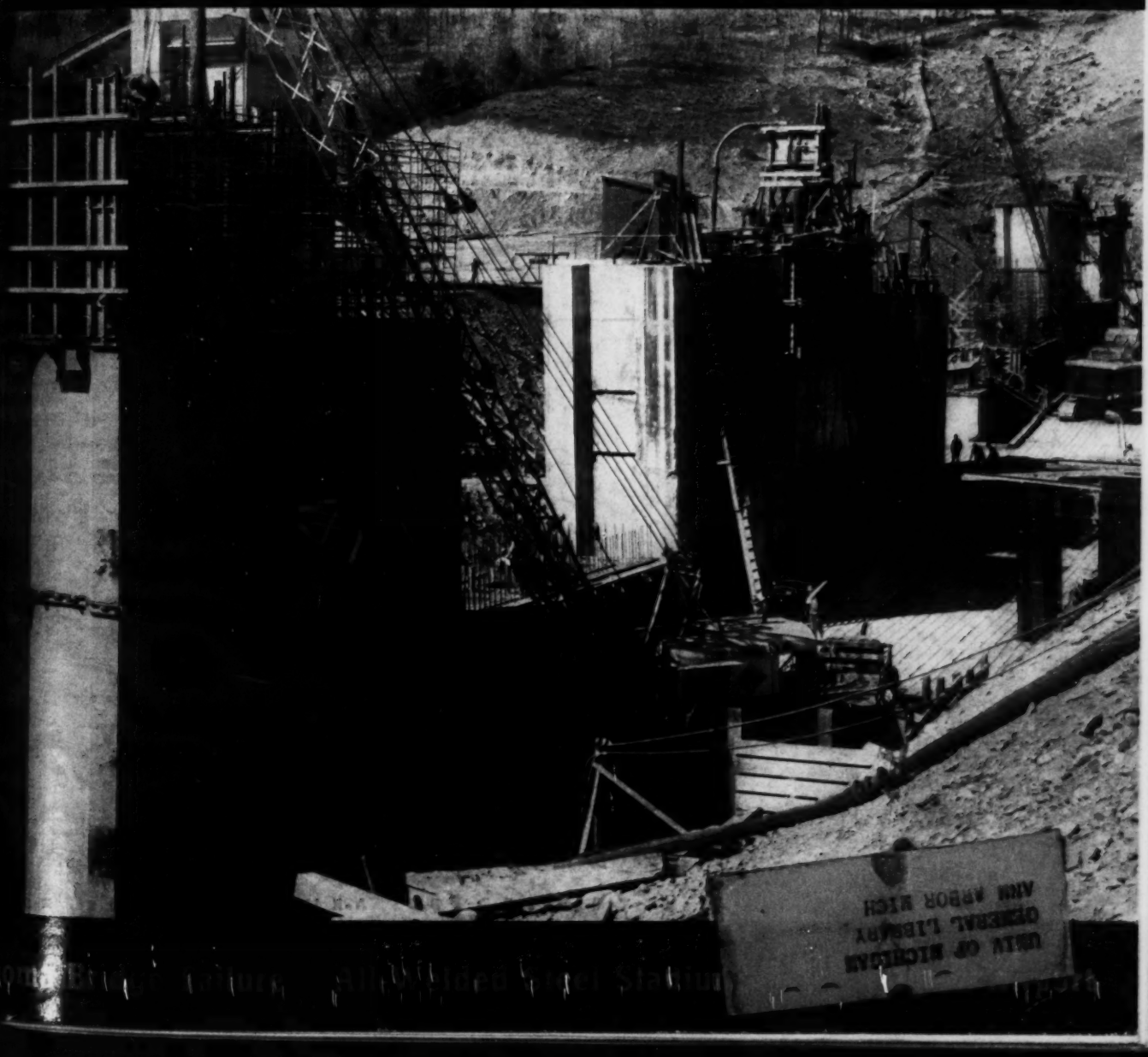
ENGINEERING NEWS-RECORD

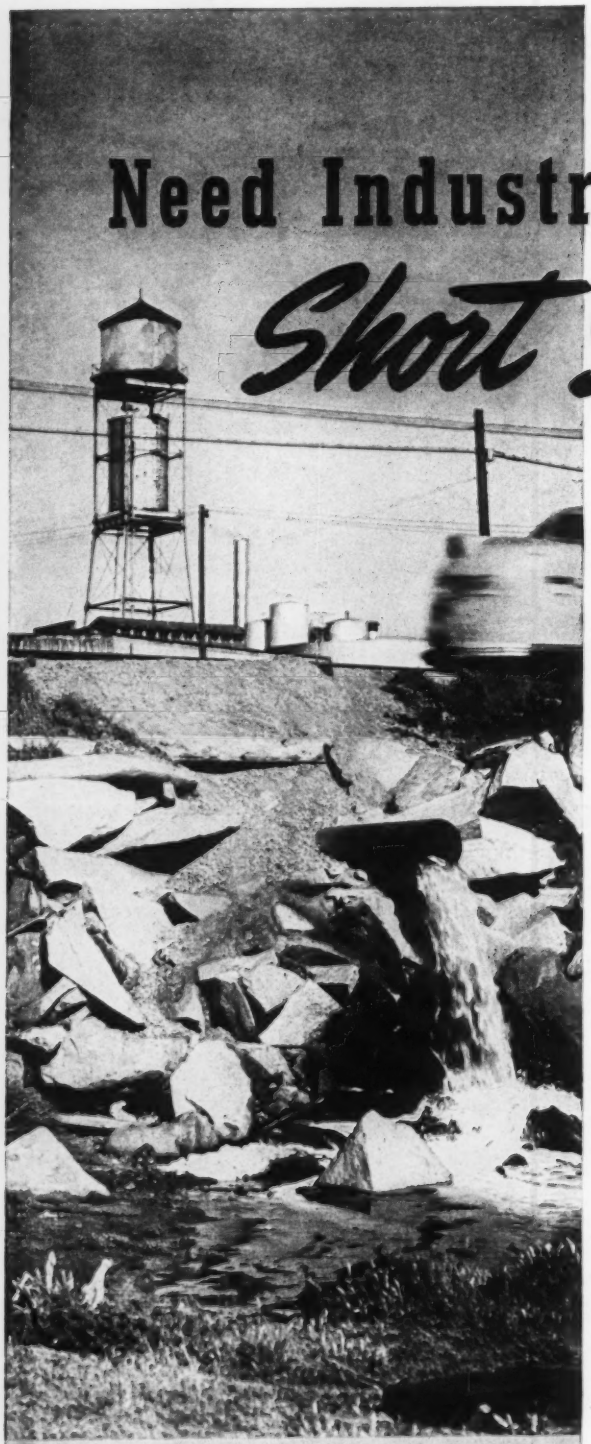
NOVEMBER 21, 1940

NOV 23

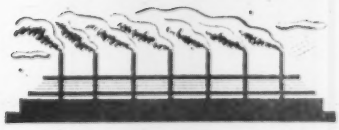
rete caissons sunk 136 ft. to rock form cutoff for Merriman Dam in the Delaware Aqueduct System.



Need Industrial Waste Cause *Short Sewer Life?*



Installed under a main line railroad near the seacoast, this ARMCO Asbestos-Bonded Pipe discharges corrosive waste from a large industrial plant. A stern test of material durability and structural strength.



It's true that sewer materials often fail quickly when handling industrial wastes. Yet there *is* a way to extend sewer life under these conditions. A special bituminous coating has proved excellent protection against corrosion. The only problem was how to obtain positive adhesion between coating and sewer.

After years of research ARMCO engineers found the answer in an asbestos-bonding process. A layer of asbestos felt is used to permanently bind the full bituminous coating to the galvanized pipe. This acid-resisting coating on ARMCO Sewer Pipe assures complete protection against corrosion. And a thick, smooth pavement of the same material safeguards the bottom from erosion. ARMCO Pipe also has the advantage of great structural strength, derived from its flexible, corrugated design—proved in more than 34 years of service.

Thus you can readily see why ARMCO Asbestos-Bonded Sewer Pipe lasts longer and costs less to maintain even in the severest service. And since ARMCO Pipe can solve your *worst* sewer problems, naturally it will be that much more durable under ordinary conditions. Try ARMCO Asbestos-Bonded Pipe on your next sewer project. You'll be repaid by years of useful, unfailing service. Why not write us about your potential needs or see the local ARMCO man. ARMCO DRAINAGE PRODUCTS ASSOCIATION, 5045 Curtis Street, Middletown, Ohio.

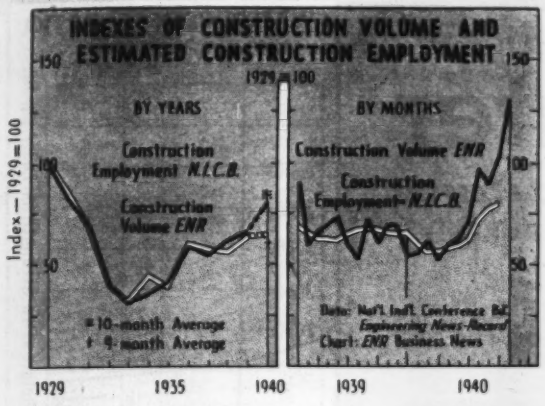
ARMCO ASBESTOS-BONDED PIPE

ORIGINATED AND DEVELOPED BY ARMCO ENGINEERS

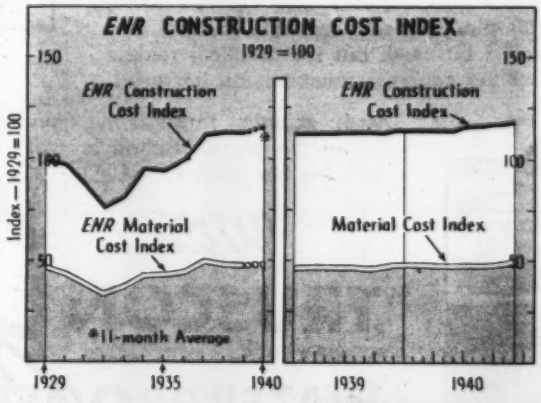
CONSTRUCTION TRENDS

Notes on significant movements in the construction industry

Record construction boosts employment



MORE THAN $2,000,000,000 in engineering construction has gone under contract or force account work in the first twenty weeks of the second half of 1940. This unprecedented speed-up in construction has been accompanied by the sharp upturn in employment and moderate increases in wage rates and material costs, shown graphically in the charts on this page.

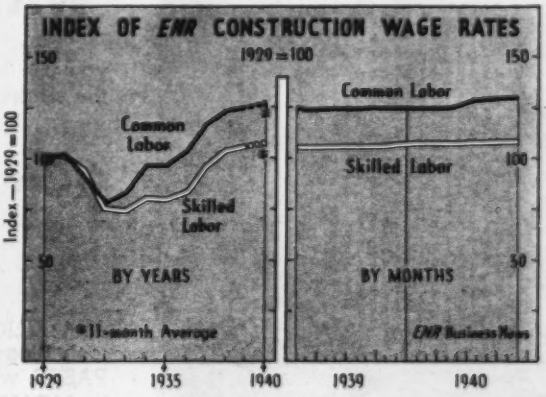


This record construction pace boosted the *ENR* Construction Volume Index (1929 = 100) to 155 for October, a new all-time high. This index measures physical volume, dollar volume adjusted for cost changes. Even though the current year's dollar total may top the former (1929) record, the increase in costs from 1929 to 1940 means that the physical volume is not expected to exceed the 1929 record. The 10-month 1940 physical volume average, for example, is 19 per cent under the 1929 construction volume index, while the dollar volumes are about equal.

The sharp upturn in construction brought about by defense awards has stimulated construction employment to show a gain of 33 per cent in September over the January total. At the opening of the year it was estimated by the National Industrial Conference Board that 1,994,000 workers were employed in the industry. The figure dropped to 1,861,000 in March, but since that time has climbed steadily to 2,658,000 employed in September. This latest estimate is 7 per cent under the yearly average of construction employment in 1930, and 20 per cent below the 1929 average, but substantially higher than all annual averages since those years.

Construction costs, the important factor in measuring physical volume, are currently 19 per cent higher than the average for 1929, and higher than at any time since the end of 1920. Costs of materials used in computing the *ENR* Construction Cost Index are up 4 per cent from 1929, and the labor component of the index is up 30 per cent.

Wage rates for skilled and common construction labor for November (*ENR* 20-cities' average) are at the highest levels ever reported.



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IT IS SO SIMPLE TO OBTAIN . . .

Better Concrete

Use Less Water

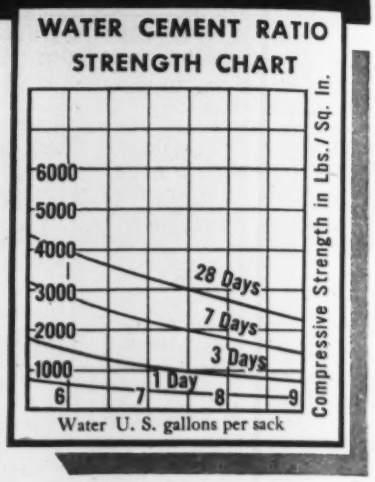
AND APPLY THE ESTABLISHED KNOWLEDGE AND ACCEPTED PRACTICE OF THE LAW OF WATER CEMENT RATIO

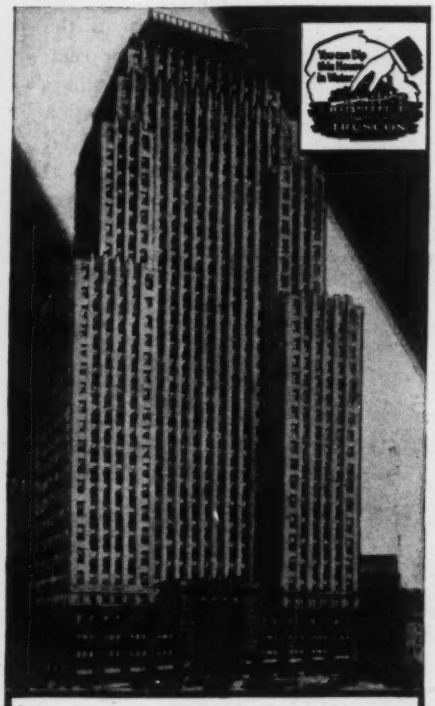
Water is incompressible—water is volatile.

Water always occupies the same amount of space, and can't be compressed.

On evaporation, water leaves millions of tiny spaces in the concrete mass.

With the application of these simple physical laws it is obvious that for better results less water should be used. But for practical results the plasticity and workability of the concrete must not be impaired.





KOPPERS BUILDING, Pittsburgh, Pa.
Graham, Anderson, Probst & White, Archts.
E. P. Mellon, Advisory Archt.
All foundations and sub-floors waterproofed with TRUSCON in 1928.

Again science, through the development of a new conception of the physical chemistry of Portland cement concrete, provides the answer.

With **TRUSCON** *Zilicon* **WATERPROOFING PASTE**

you use less water without impairing but actually improving the workability of the concrete.

you get –

- **Increased compressive strength.**
- **Reduced shrinkage.**
- **Marked increase in resistance to freezing and thawing.**
- **With fixed water cement ratio an increase of over 100% of slump.**

And ZILICON, the new chemical term in Portland cement technology—combined with TRUSCON WATERPROOFING PASTE, with its thirty years of established use and prestige—

NOW OFFERED WITH A 25 YEAR GUARANTEE

Write for Bulletin No. 494

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F. E. SCHMITT *and* WALDO G. BOWMAN, *Editors* • ALBERT E. PAXTON, *Manager*

Vol. 125 NOVEMBER 21, 1940 No. 21

THIS WEEK

• The wide variety of subjects treated in this issue of *News-Record* should have an appeal to every branch of the construction industry—unusual design problems, new construction methods, and advanced thoughts on plant design, equipment and operations—all presented in a manner for quick and easy reader consumption.

• The failure of the Tacoma Narrows Bridge has not only attracted the attention of engineers and construction men the country over but the public as well. For engineers who wish to look beyond the sensational aspects of the failure to learn the probable cause and the lessons that the failure teaches, there is given in this issue a survey by the editors of all pertinent facts.

• Electric welding of steel members continues to play a big part in the speed-up demands on the construction industry. The application of welding to large and small units and for all types of structures is reducing the cost of former construction methods. In this issue a stadium seating 10,000 and a 23-story hospital building affords excellent examples of what is and can be done through this medium.

THINGS TO COME

• Air conditioning as an essential element in the modern building presents an ever widening influence upon the design and construction of all types of buildings. New problems are being presented as the application of air conditioning expands and new solutions are being found. One of the most interesting is how the structural design of the 16-story Esperson Building in Houston, Tex., was designed around the air conditioning system as the controlling factor. This project is described in the Dec. 5, issue.

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Number of copies of this issue printed: 34,817

NO MORE CURB BLACKOUTS EVEN ON RAINY NIGHTS

with new White Reflecting Curb

THE DARKER the night and the wetter the pavement, the greater is the driving visibility on roads guarded by White Reflecting Curb made with Atlas White cement. Under rain, the visibility of smooth concrete curb and pavement drops to practically zero. But on main New Jersey highways today, a new type curb with white concrete face and reflecting saw-tooth design shows up even brighter on rainy nights than in clear weather—*under no other illumination than ordinary car headlights.*

Moderate in first cost, less expensive in upkeep, and as permanent as the road itself, this simple but scientific contribution to traffic safety pays for itself in short time.

Atlas White cement is used for White Reflecting Curb because white reflects the maximum amount of light and gives greatest contrast to surrounding pavement by day and night. For further information, write Universal Atlas Cement Co. (United States Steel Corporation Subsidiary), Dept. C3, Chrysler Bldg., N.Y.C.

All photos by New Jersey State Highway Department

FIG. 1 Smooth White Curb—Dry and Wet

FIG. 2 White Reflecting Curb on rainy night

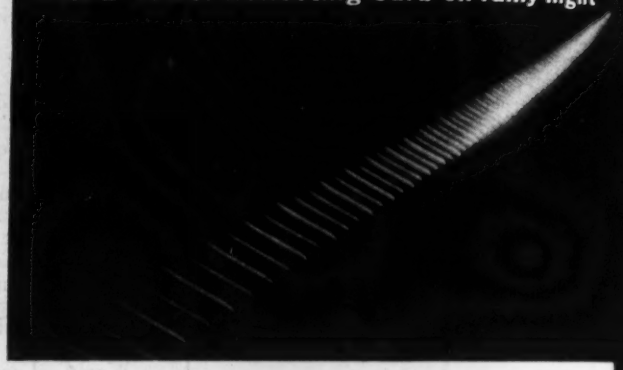


Fig. 1. Here is a strip of ordinary smooth white concrete curb under car headlights at night. It's only slightly more visible than the road. And when water (shown by arrow) covers the curb and road, there's a complete blackout. The curb disappears.

Here's the reason: Smooth curb and road reflect most headlight rays forward, away from the driver; visibility is poor. On a rainy night, water fills the slight irregularities of smooth curb and pavement, causes them to act even more like a mirror, and to reflect even more headlight rays forward, away from the driver; visibility is nil (shown by arrow). Road and curb are almost completely obscured.

FIG. 3

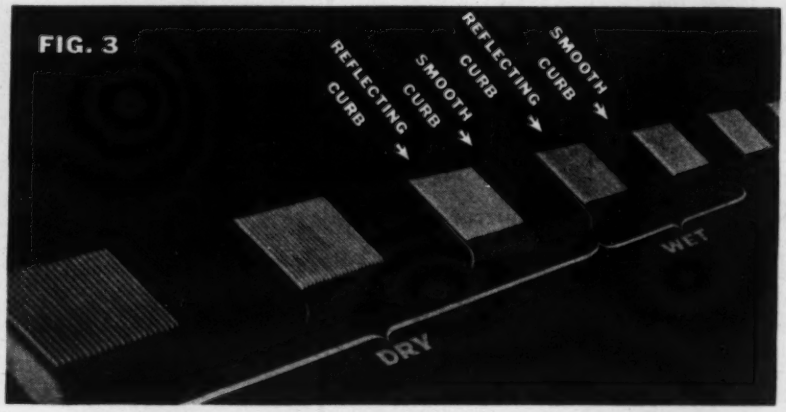


Fig. 3. A New Jersey State Highway Test: Alternate sections of smooth white curb and reflecting white curb under dry and wet conditions as seen by night drivers, under car headlights only, with no other illumination. All sections, the smooth ones which appear dark, as well as the reflecting sections which appear white, are made of identical white materials.

White Reflecting Curb made with Atlas White cement provides a curb that defines the road by day, is highly visible on dry nights, and even more visible on wet nights.

Fig. 2. White Reflecting Cur makes a gleaming ribbon of lig extending far in front of the ca Even under unfavorable cond tions—on a rainy night, as he —the curb itself seems to b lighted. But actually this is sim ply due to the light reflected fro the driver's headlights—no oth illumination.

The reason is simple: Smoo curb wastes the headlight ray reflects them forward, away fro the driver; visibility is poor. B the specially designed saw toothed faces of White Reflectin Curb conserve the headlight ray catch and reflect them back t the driver; visibility is high. An on wet nights the film of water o the reflecting faces intensifies th reflection — illumination is eve brighter—visibility is even high —night driving is safer.

ENR-M-17

WHITE CONCRETE REFLECTING CURB

MADE WITH ATLAS WHITE CEMENT

Storm causes wide damage to cities around Lake Michigan

Record low water in the lake tests new locks at mouth of Chicago River and drains most of Calumet Lake into Lake Michigan. No serious damages to water supply

A violent wind and rain storm sweeping in from the Mississippi Valley on Nov. 11 caused wide damage to property around Lake Michigan and blew the waters of the lake away from the south and east shores, seriously threatening Chicago's water supply with pollution. Many lives were lost in boats and ships that foundered in the storm. In the upper Mississippi Valley and the lake region the rain turned to snow tying up rail and road transportation.

Low water at Chicago

Normal lake level at Chicago is –0.7 ft. on the Navy Pier gage. At 6:10 p. m. on Nov. 11 it had dropped to –4.8 ft. The previous low was a gage reading of –4.01 ft. on Jan. 23, 1926.

The locks controlling the mouth of the Chicago River got their first real test in Monday's storm and proved their worth in preventing pollution of the lake water at the waterworks intake cribs that would otherwise have resulted from a reversal of the flow of the river. For more than three hours the water on the river side of the gates stood 3 ft. higher than on the lake side. The gates are designed for a differential of 8 ft. in head on either side. The Sanitary District of Chicago had completed the job of sealing bulkheads connecting the locks with the shore, so no river water escaped.

Calumet River reverses flow

At the south end of the city, however, the Calumet River reversed its flow, taking out most of Calumet Lake and poured into Lake Michigan at a maximum rate of 2,500 cfs. Due to the fact that the wind by that time had shifted to the southwest, the river water was carried to the east of the 86th St. water intake crib, thus preventing serious pollution of the water supply.

The wind, starting from the southeast, switched to south and then to the southwest as it approached maximum velocity, reaching 58 mph in sustained 5-min. periods. Momentary gusts as high as 66 mph were reported. The barometer reached a low of 27.9 at 11:15 a.m.

Property damage on shore was heaviest at Chicago and Milwaukee. A huge electric sign, facing Grant Park in Chicago, rated as the largest in the world, was demolished by the wind.

East and north shore reports

To check the damage on the east and north shores of Lake Michigan, *Engineering News-Record* asked several city engineers and managers for reports on the effect of the storm in their localities. All reported severe lake damage, loss of life and shipping, and some damage to shore property.

The City of Manistique, on the north shore of Lake Michigan, received the

U. S. Engineer photo

Work advances on $4,170,000 Nimrod Dam in Arkansas

With 10,000 cu. yd. of concrete placed during the month of October, the $4,170,000 Nimrod Dam project for flood control on the Fourche La Fave River 4 miles west of Nimrod, Ark. is rapidly taking shape. Completion is scheduled for March, 1942. The dam, being built about 60 miles above the confluence of the Fourche La Fave with the Arkansas River, is of the gravity type and will control a drainage area of 680 sq. mi. Pool storage is to be 30,000 acre-ft. with flood control storage of 305,000 acre-ft. The dam is designed for an ultimate storage of 640,000 acre-ft. to be obtained by raising the height of the dam and adding piers and spillway crest gates.

The computed average annual runoff above the dam site over a 22-year period is 640,000 acre-ft. or an average rate of 885 cfs. The maximum computed monthly runoff of 587,000 acre-ft. occurred in April, 1927, with an estimated peak discharge of 51,000 cfs.

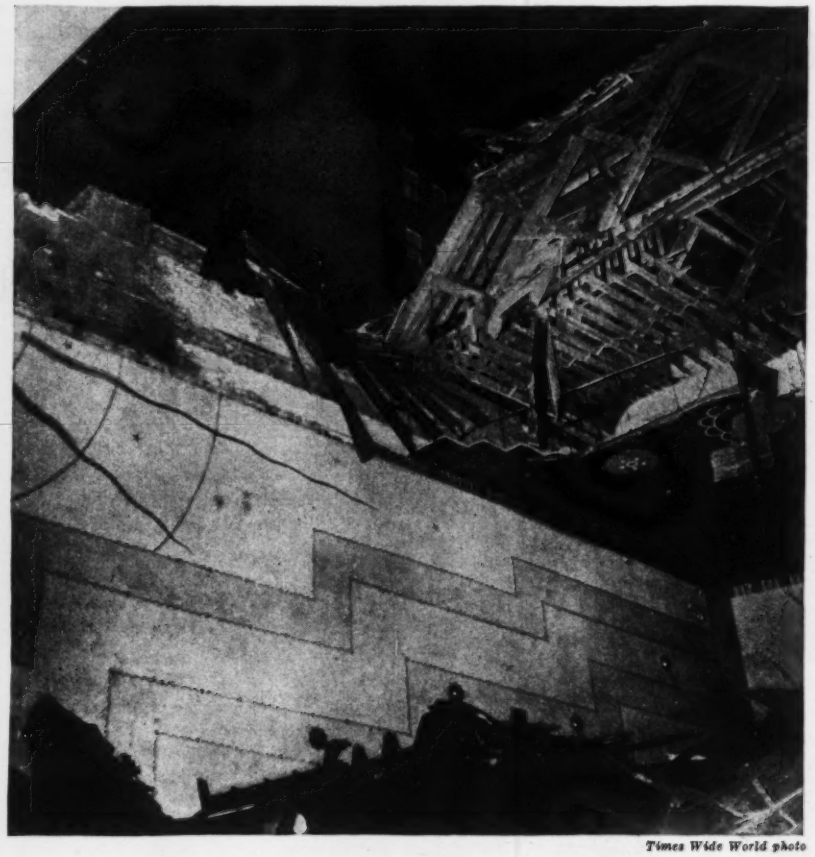
The dam, which is to contain about 115,000 cu. yd. of concrete, will have a height of 100 ft. above stream bed.

The $1,100,689 general contract, held by Russ Mitchell, Inc., and Brown & Root, Inc., Houston, Tex., was 23 per cent complete Nov. 5. H. S. Hurlburt is supervising the work and W. W. Ralphe is resident engineer for the U. S. Engineers.

brunt of the storm, according to City Manager P. H. Beauvais. The water at one time rose 4 ft., inundating Lakeside Park and U. S. Highway No. 2, which is 1,000 ft. inshore. The wind attained a velocity of 78 mph causing considerable damage to buildings and local shipping. Though telephone and power lines were badly crippled, power was available at the city pumping plant at all times.

Seymour Justima, city manager of Grand Haven, Mich., on the lower eastern shore, reports that the wind reached a peak of 67 mph at 9:15 p.m. Monday, and the lake level dropped 0.8 ft. Many cottages along the shore were damaged and sand formed in high drifts along the beach.

At South Haven, Mich., near the lower end of the lake, S. H. Smith, city engineer, reports that the waterworks intake was not damaged, though two fishing boats and eight men were lost just offshore.



Times Wide World photo

Old wooden theatre roof falls in Pittsburgh

Two wooden roof trusses in the 50-yr. old Harris-Alvin theatre on South St. in Pittsburgh, Pa., collapsed at 6:23 p.m. on Nov. 14, dropping the ceiling and roof on empty seats in the forward part of the auditorium, now used as a motion-picture house. No one was injured by the fall due to the fact that plaster began to drop from the ceiling some time before the collapse occurred and ushers had moved the 200 patrons back under the gallery. Three people were injured in the stampede that followed. Small pieces of plaster began to fall from the ceiling soon after 5 p.m., according to press reports, and increased in size as time went on.

The theatre was built in 1891 by Charles L. Davis. Later it passed to Paul Keith who left it to Harvard University, the present owner, on his death. It is a brick-bearing-wall structure with wooden roof trusses of 80-ft. span. In 1934 the interior was modernized and the balcony was strengthened but the wooden roof trusses were not replaced. No appreciable addition was made to the load on the roof trusses during the modernization, according to H. L. Seaman, superintendent of the inspection department of the city's bureau of buildings.

Andrew Frazier, city building inspector, is quoted in the Pittsburgh "Post-Gazette" as having stated that the building was inspected from roof to cellar on Oct. 25 and found entirely safe. Other press reports state that the roof trusses were examined 14 months ago, but no information is available as to conditions found as all building inspection reports have been sequestered by Public Safety Director E. A. Fairly pending an investigation of the collapse, which has been ordered by city authorities.

H. J. Lewis, city engineer of Muskegon, Mich., about midpoint on the eastern shore, reports that no damage was done to the Muskegon filtration and pumping plant located on the shore of the lake. The water rose 4.2 ft. at the pumping station wet-well on the shore, the turbidity rose 25 ppm. and the color of the water changed from green to gray. Waves offshore were reported 30 ft. high. The wind reached a maximum velocity of 67 mph. The waterworks intake, located 37 ft. below normal water level and protected by a seawall, was not damaged. A pavement along the shore was undermined, the shore line was changed 150 ft. in places, and 3,000 ft. of snow fence used to control drifting sand was lost.

A wind velocity of 75 mph was reached at Ludington, Mich., near the north end of the lake on the eastern shore, reports Charles W. Baggott, city engineer. No damage was done to shore front property, breakwaters or the waterworks intake. In contrast to the high rise in lake levels reported elsewhere along the northeast shore, the water rose a maximum of only 18 in. above normal at Ludington, most unusual for the wind velocity and direction, according to Baggott. Property damage in the town was confined to loss of roofs and glass.

• • •

Penn Turnpike tolls $233,829 first month

Reports on the first month of operation of the Pennsylvania Turnpike show that 264,480 motor vehicles used the superhighway during that period with toll collections totaling $233,829. This is $17,559 more than the engineers' estimate.

Each vehicle paid an average of 88 cents in tolls, which shows that a considerable number of the users of the road do not go the entire length of the 160-mile superhighway as the lowest toll, that for passenger cars, is $1.50 for a full-length trip.

While passenger traffic exceeded estimates, the Turnpike Commission was disappointed because only 14,000 trucks used the road during the month.

• • •

Federal highway aid shut off in Georgia

W. L. Miller, chairman of the Georgia State Highway Board, has been notified by the federal government that no further federal aid to Georgia for road work would be approved until that state is able to meet its road obligations promptly. State Auditor Arnold of Georgia reported to Governor Rivers recently that the Georgia Highway Department ended the fiscal year on June 30 with nearly a $14,000,000 deficit.

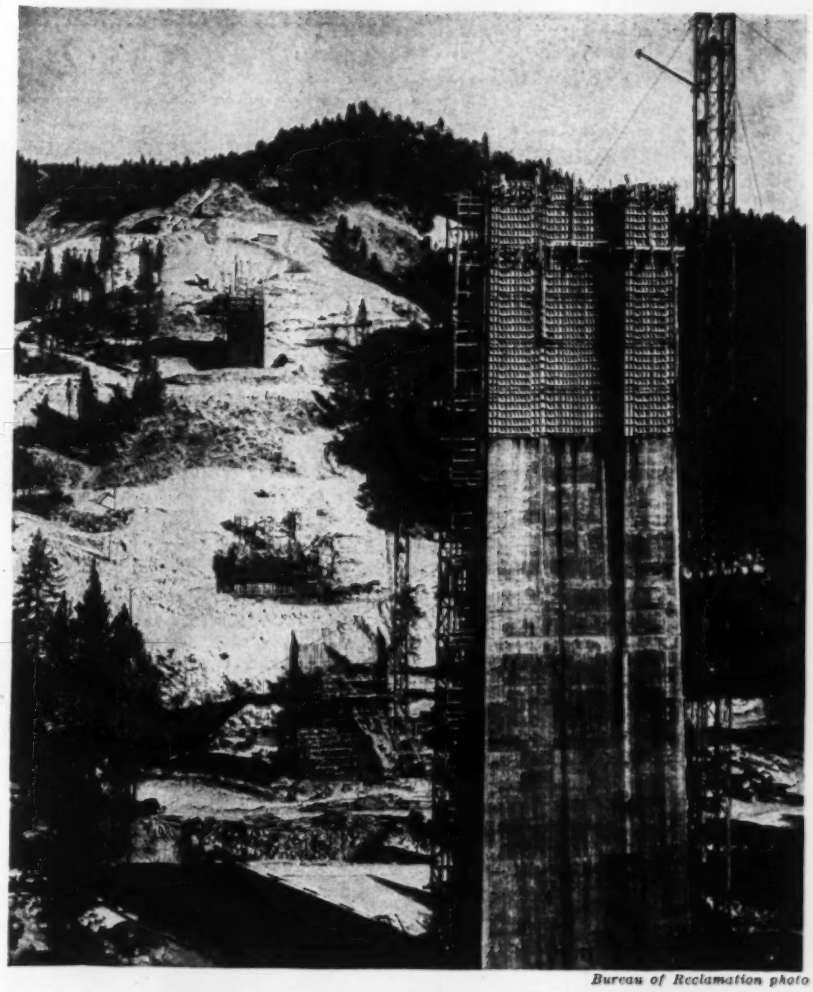
WASHINGTON HIGHLIGHTS

PUBLIC STORAGE and warehouse facilities are adequate for immediate civil and military needs, though there may be need for new private facilities in some industries. However, if the war and the defense program are long-continued, serious storage problems may develop. This was the consensus of a panel discussion on warehousing held in connection with the conference of wholesale distribution called by the Defense Commission. It was recommended that the government not build new warehouses for defense needs until all vacant rentable space is utilized. A survey of available warehouse facilities to determine what construction may be necessary is now being carried out by the transportation section of the Defense Commission.

CONSTRUCTION of underground parking areas deep enough to be used if needed as air raid shelters is being studied by the housing coordinator's office.

RECENT LUMBER price increases are blamed on a poor buying policy on the part of defense agencies by the Department of Commerce's lumber survey committee in its quarterly report this week. Sudden and heavy requirements, comparatively restrictive specifications, requests for bids on short notice for quick deliveries on large quantities, and acting on a highly responsive market resulted in the price rises, it is stated. Army troop housing will require about 1,200,000,000 board feet by July 1942, the committee estimates; much of this has already been purchased. Defense housing will require, within the next eight months, about 500,000,000 ft. and Navy will need 400,000,000 ft. In addition, an indeterminable quantity will be needed for munitions plants, warehouses, crating, hangars and the like. Lumber consumption during the fourth quarter of 1940 is estimated at 7,342,000,000 fbm, and for all of 1940 at 28,500,000,000 fbm—the greatest since 1929. Production will total 27,500,000,000 fbm. Mill stocks Oct. 1 were 7,160,000,000 fbm. or 6 per cent below July and 8 per cent below a year previous.

FHA-INSURED mortgages and pending mortgages have now reached $2,800,000,000 and it has therefore become necessary to take advantage of the clause in the law which provides the three billion limit on commitments may be increased to four billion. Last week President Roosevelt authorized such an increase.



Bureau of Reclamation photo

World's tallest bridge piers take shape

Well advanced is the work of constructing one pier 358 ft. high, another nearly as tall, and several shorter piers for the combined $5,000,000 railroad and highway bridge being built across the Pit River arm of the Shasta Dam reservoir in California. The two taller piers when complete will be the tallest in the world; the higher one has a 90x95-ft. base. During construction, concrete in the piers is cooled by a water circulation system.

The lower bridge deck will carry a double track for the Southern Pacific main line; on the upper deck, 530 ft. above stream bed, a 40-ft. roadway flanked by two 2½-ft. sidewalks to take care of traffic on U. S. Highway No. 99 will be provided.

The central span of the bridge, a cantilever truss, is to be 630 ft. long. There will also be two 497-ft., three 282-ft., and two 141-ft. truss spans. The girder spans will include one 150 ft. and four of 141 ft. Overall length of the structure will be 3,588 ft.

Union Paving, San Francisco, with a $1,138,000 contract, is building the piers and abutments. The American Bridge Co. will erect the superstructure, under the terms of a $2,588,000 contract. Title to the bridge will remain with the U. S. Government, but California is to have a perpetual easement for highway uses and maintenance.

For a sketch of how the bridge will appear when complete see ENR *Jan. 25, 1940, p. 139.*

North Carolina road engineers get more pay

The state of North Carolina has found it necessary to raise the pay of its highway engineers to prevent them from resigning to take jobs at higher salaries in private employment or on national defense projects. After officials of the highway department had complained that between 30 and 40 per cent of their engineers had resigned within the past few months and that they could not keep the others from leaving unless salary increases were provided. Governor Hoey approved the suggested pay increases and instructed the state's budget bureau to work out the details.

Milwaukee to use sand from lake for paving

Ordinary Lake Michigan sand, available at $1.25 per ton, will be used by the city of Milwaukee, Wis., for asphalt paving instead of a special silicate sand, costing $2.38 per ton, which has been used for the past several years. Several months ago the city central board of purchases decided to try out the lake sand, and R. W. Gamble, superintendent of street construction, reported to the board recently that the lake sand had proved satisfactory.

• • •

Bids opened Dec. 4 for Panama Canal excavation

Lieut. Col. Thomas B. Larkin, supervising engineer of the special engineering division of the Panama Canal work, reports that specifications for bids on the dry excavation for the third set of locks are now available and that bids are to be opened for the work Dec. 4. Included is the removal of nearly 12,000,000 cu.yd. of earth for the locks and approaches.

Initial work on the new Gatun locks was begun Sept. 9, when a force of workers began clearing some 500,000 acres of ground in the construction area.

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Building owners file suits against Chicago subways

The owners of the old Capitol building, at Randolph and State streets, Chicago, which was torn down instead of being underpinned in advance of subway construction on State St., have filed suit for $850,000 against the city of Chicago for alleged damages resulting from the subway construction. The nineteen-story building erected in 1891, was torn down last year and replaced with a two-story taxpayer. The owners claim a fair value of the old building was $750,000 and that an additional $100,000 is due them for use of their property by the subway structure. They alleged that title to the area between building and the centerline of State St. formerly belonged to the original owners, and that only surface rights were deeded to the city when the street was laid out in the early days.

Other large suits pending in the courts include that of the Monadnock building for $210,000 for the cost of underpinning (*ENR* Sept. 26, 1940, p. 422) and the 140 North Dearborn Building for $300,000 damage and restoration cost. Numerous smaller suits have been filed, bringing the total up to $1,500,000. None of the suits have been decided in any court.

JOBS OF THE WEEK

REPLACEMENT CENTER, Camp Lee, Va.

Doyle & Russell, Richmond, and Wise Constructing Co., Richmond, were awarded a $7,539,051 joint contract for constructing the replacement center at Camp Lee, Fairfax County, Va. for the War Department.

CANTONMENT, Camp Wolters, Mineral Wells, Texas

Cage Bros., Austin, and F. M. Reeves & Sons, Bishop, are constructing a camp replacement center at Camp Wolters under terms of $5,628,000 contract with the Constructing Quartermaster, Washington, D. C.

HOUSING, Fort Knox, Ky.

Whittenberg Construction Co., and Struck Construction Co., will construct the housing at Fort Knox. George M. Eady Co. and Highland Co., were awarded the contract for the sewers, excavation and street work. Estimated cost of the project is $5,000,000.

TNT AND DNT PLANT, Weldon Springs, Mo.

Fraser Brace Engineering Co., New York, got the contract for constructing a TNT and DNT plant at Weldon Springs, Mo., for the War Department, Washington, D. C. Atlas Powder Co., Wilmington, Del. will operate plant, which is estimated to require $15,000,000 for construction.

WAREHOUSES, Ogden, Utah

Al Johnson Construction Co. and Leck Construction Co., Minneapolis, Minn., will construct seven warehouses at Hill Field for the Constructing Quartermaster under terms of a $1,492,000 contract.

NURSES QUARTERS, Canal Zone.

MacDonald Construction Co. and G. L. Tarlton Contractors, Inc., both of St. Louis, were awarded a $4,115,478 contract for constructing the nurses home and hospital sector at Fort Clayton and Fort Gulick for the Constructing Quartermaster, Albrook Field.

REPLACEMENT CENTER, Nacimento, Calif.

Ford J. Twaits Co., Los Angeles, Calif. and Morrison-Knudsen Corp., Boise, Idaho, received a $6,018,753 contract for constructing a replacement center.

AIRPLANE PLANT, Santa Monica, Calif.

Douglas Aircraft Co., Inc., Santa Monica, Calif., and its subsidiary, the Western Land Improvement Co., will construct an airplane plant according to terms of a $11,254,700 contract on a cost-plus-fixed-fee basis with the War Department, Washington, D. C.

HOUSING PROJECT, Bremerton, Wash.

Western Construction Co., Seattle, Wash., was low bidder for constructing the defense housing near Puget Sound Navy Yard with a bid of $1,566,000. Bremerton Housing Authority took bids Nov. 13 for the job, which is a USHA project.

PIER, Puget Sound, Wash.

Puget Sound Bridge & Dredging Co. and Rumsey & Co., Seattle, are constructing Pier No. 3 at Puget Sound Navy Yard under terms of a $2,000,000 contract with Bureau of Yards & Docks, Navy Department, Washington, D. C. The contract was awarded on a cost-plus-fixed-fee basis.

ASSEMBLY PLANT, Detroit, Mich.

O. W. Burke Co. will construct an assembly plant, a 1-story assembly building and 4-story office building for the Packard Motor Car Co., Detroit. The work is estimated to cost $1,500,000.

JETTY, Louisiana

Massman Construction Co., Kansas City, Mo., was low bidder with a bid of $1,129,200 for repairing an existing rubble mound jetty at Calcasieu Pass. Work involves 220,000 tons of stone. U. S. Engineers, Second New Orleans District, took bids Nov. 7.

WAREHOUSE, Philadelphia, Pa.

Day & Zimmerman, Inc., Philadelphia, Pa., was awarded a $1,170,000 contract on a cost-plus-fixed-fee basis for constructing a warehouse and accessories at Marine Corps depot, Philadelphia Navy Yard, by the Bureau of Yards & Docks, Navy Department, Washington, D. C.

TEMPORARY REPLACEMENT, Hitchcock, Tex.

Nathan Wohlfeld, E. L. Dalton Co. and Dellone Co., of Dallas, were awarded a $2,960,000 contract for constructing the temporary replacement camp near Hitchcock for the Constructing Quartermaster, Washington, D. C.

NOTE—Additional bidding and contract news on over 800 projects large and small, appear in the Construction News Section beginning on page 119.

Fairchild Aerial Survey photo above

New York's Queens-Midtown tunnel opened

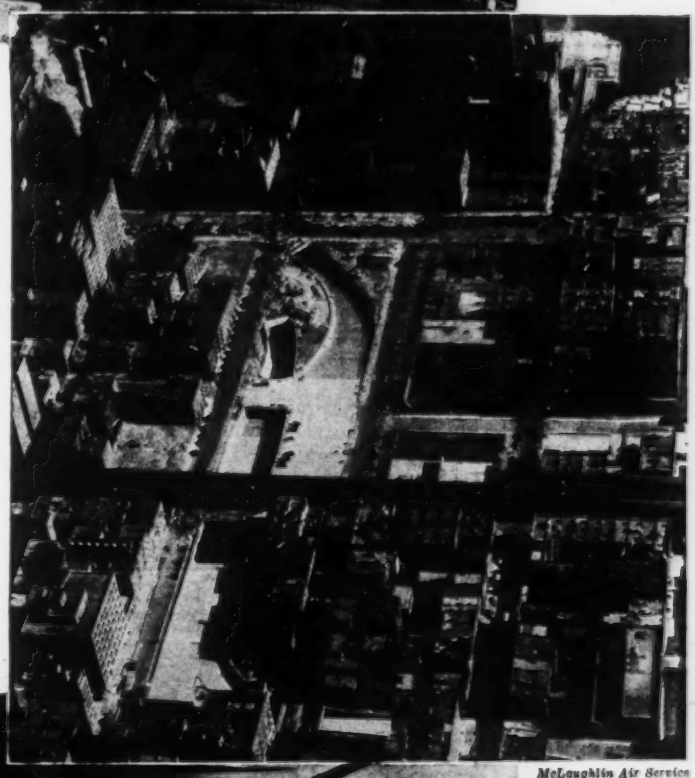
Providing for two lanes of traffic in each direction by twin 31-ft. tubes, New York City's $58,000,000 Queens-Midtown Tunnel connecting the Island of Manhattan and the Borough of Queens on Long Island, was dedicated and opened to traffic Nov. 15. It is the first vehicle tunnel under the East River.

The Manhattan plaza, which is shown in the right photo with the entrance at the right, occupies most of the blocks bounded by 36th and 37th Sts. and 1st and 3rd Aves. New access streets have been cut parallel to the north-south avenues. The Queens plaza, where the toll booths are located, is shown in the upper photo along with the elevated approach road. The lower photo is an interior view of one of the tubes.

The work was done by the New York City Tunnel Authority, Ole Singstad, chief engineer; Walsh Construction Co. constructed the twin tubes.

Additional details of design and construction are given in ENR Dec. 21, 1939 p. 830 and Jan. 4, 1940 p. 29.

McLaughlin Air Service



McLaughlin Air Service

Wide World photo

OBITUARY

WILLIAM H. SHELTON, 80, civil engineer, died Nov. 11 at Orange, Va. He was a former city engineer of Dunkirk, N. Y., and he had been employed on harbor work on Lake Erie and railroad construction in West Virginia.

JOHN JAMES MCKEE, 61, contractor and builder who built many barracks in various parts of the United States during the World War, died in Columbus, Ohio, Nov. 10. He was engaged in the contracting business in Detroit, Chicago and Cleveland before going to Columbus three years ago.

EDGAR S. BRUBAKER, 52, civil engineer formerly with the Allegheny County (Pa.) road department and the Pennsylvania State Highway Department, died recently at Wilkinsburg, Pa.

JAMES L. PARKER, 66, general contractor at Marietta, Ga. for 31 years, died last week.

FRANK D. BARDWELL, 71, for 37 years a civil engineer in the grade crossing department of the New York Central Railroad, died Nov. 11 in Springfield, Mass.

SCHUDDER HART ROBERTSON, 76, retired builder and for more than 20 years an inspector in the department of highways and sewers for the city of Pittsburgh, died Nov. 12 in that city.

P. B. JENNINGS, 60, construction superintendent for the Sorenson Construction Co., Dallas, Tex., died Nov. 10 as a result of an accident on a job at Albuquerque, N. M.

HARRY FERGE, superintendent of the Northern Construction Co., died Nov. 14 at his home in Milwaukee.

JOSEPH A. FAY, for 15 years superintendent of the Southern California Water Co., died Nov. 9 in Los Angeles.

ERNEST B. MCCREADY, operator of a testing laboratory at Allentown, Pa., and a former member of the faculty at Temple University, died in Allentown Nov. 13 at the age of 68. Following graduation from Lafayette College in 1898 with a master's degree, McCready was employed as instructor at New Hampshire College and as assistant chemist with the New Hampshire Experiment Station. He then taught at Temple and later joined the New York Rapid Transit Railroad Commission as assistant engineer on inspection of materials. From 1905 to 1910 he was general manager of the Lehigh Valley Testing Laboratory.

ROBERT K. PALMER, former chief engineer and vice president in charge of operations of the Hamilton Bridge Co., Hamilton Ont., died Nov. 17.

GEORGE F. TURNER, 58, former city engineer of Youngstown, Ohio, died in that city Nov. 12. Before serving with the city of Youngstown, Turner worked as assistant county surveyor of Mahoming County. For a time he was employed by the Pennsylvania R. R.

W. R. JENKINS, 62, highway commissioner of Green Lake County, Wis., for the last ten years, died Nov. 6.

Milwaukee loses in water bill fight

The Wisconsin State Supreme Court ruled Nov. 15 that the city of Milwaukee could add only about $80,000 to the water bill of the suburb of West Allis for use of water since 1930. The city has sued for $546,000 and in a lower court had won the right to collect 9 cents per 100 cu.ft. from the suburb, which had previously paid 6 cents and refused to pay more. No rate was on file with the public service commission during the ten-year period and a commission has finally fixed the price at 6½ cents.

CONTRACTS AND CAPITAL

ENGINEERING CONSTRUCTION awards for the short week due to the Thanksgiving Day holiday total $74,657,000. This is 57 per cent higher than the volume for the corresponding 1939 week, but 46 per cent under last week. Awards top their respective 1939 values for the twelfth consecutive week, and despite the holiday are slightly above the 1940 average week, $74,452,000.

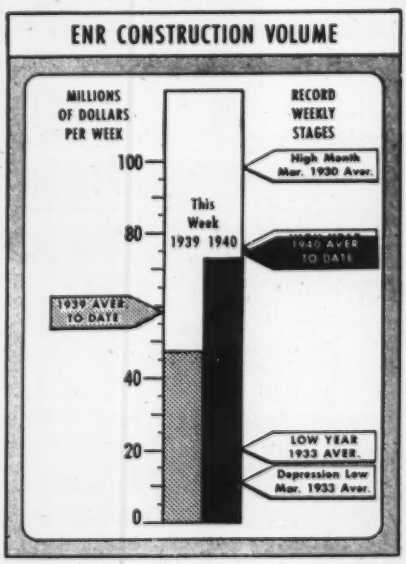
Public construction is 145 per cent above a year ago, but 43 per cent under a week ago. Private construction is 39 and 57 per cent lower, respectively, than last year and last week.

The current week brings 1940 construction to $3,499,236,000, an increase of 27 per cent over the 47-week 1939 period. Private awards are up 28 per cent, and public construction tops last year by 27 per cent as a result of the 271 per cent gain in federal work.

In the classified construction groups, gains over the 1939 week are in waterworks, sewerage, public buildings, and earthwork and drainage. All classes of work are under their respective totals of a week ago.

New capital for construction purposes for the week totals $3,850,000. This compares with $63,071,000 reported for the corresponding 1939 week. The current week's financing is made up of $1,872,000 in state and municipal bonds, $1,846,000 in corporate security issues, and $132,000 in RFC loans for public improvements.

New construction financing for the year to date, $3,600,879,000 tops the $2,333,620,000 reported for the 47-week 1939 period by 54 per cent.



CONTRACTS

(Thousands of dollars)

	Week Ending Nov. 23 1939	Nov. 14 1940	Nov. 21 1940
Federal	$3,041	$74,312	$48,212
State & Municipal	21,724	32,198	12,487
Total public	$24,765	$106,510	$60,699
Total private	22,890	32,609	13,958
TOTALS	$47,655	$139,119	$74,657
Cumulative 1940	(47 weeks)		$3,499,236
1939	(47 weeks)		$2,755,028

Note: Minimum size projects included are: Waterworks and waterways projects, $15,000; other public works, $25,000; industrial buildings, $40,000; other buildings, $150,000.

NEW PRODUCTIVE CAPITAL

	1939 47 Wks.	Cumulative 1940 47 Wks.
NON-FEDERAL	$1,205,592	$1,318,918
Corp. Securities	198,668	223,640
State & Mun.	517,905	456,670
R.E.A. loans	89,445	40,000
U.S.H.A. loans	291,647	218,704
R.F.C. loans	7,927	195,904
Fed. Aid-Hwy.	190,000	184,000
FEDERAL	$1,128,028	$2,281,361
TOTAL CAPITAL	$2,333,620	$3,600,879

FHA MORTGAGES

	Week Ending Nov. 18 1939	Nov. 9 1940	Nov. 16 1940
Selected for appraisal	$19,002	$24,348	$20,136*
Cumulative 1940	(46 weeks)		$1,153,150*
1939	(46 weeks)		$1,024,638

* Subject to revision.

ENR INDEX NUMBERS

			1913	1926
Index Base=100				
Construction Cost	Nov.,	'40	247.24	118.85
Building Cost	Nov.,	'40	206.24	111.49
Volume	Oct.,	'40	490	213

Plans agreed upon for Atlantic bases

Preliminary plans for the $500,000,000 string of island naval bases off the Atlantic coast have been agreed upon between the British and United States negotiators, except that for Trinidad. Largest of the seven other bases, will be at Bermuda; and a powerful naval and seaplane base with garrison area will be established at St. David's Island. Nearby Small Island will be developed for storage of explosives. Castle Harbor will be improved in connection with the naval base. In addition, a landplane station will be established on Long Bird Island, which will have to be enlarged by dredged fill to accommodate the runways.

In Newfoundland a 22-acre naval base will be established at St. John's harbor, with about 1,250 ft. of wharfage. There will also be an airbase and army training ground on the Argentina peninsula, and a 160-acre garrison area near Quidi Vidi Lake.

At Jamaica a fleet anchorage shall be developed at Portland Bight, together with shore facilities, defense batteries, and an auxiliary landing field. The Port Royal dock yard will be available jointly to the United States and Britain.

At St. Lucia a 120-acre seaplane base is to be built, and British Guiana will be both a seaplane base and a patrol plane squadron base.

Areas to be developed have been decided upon at Antigua Island, but there has been no announcement of the facilities to be installed. No cost figures have yet been announced.

Farther in the future but definitely on the cards is construction of air and other bases in South America. The government has denied that it plans to build bases of its own in any South American country.

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Other big defense jobs for future studied

Two big jobs of industrial building for defense which go beyond present programs are now being studied. One is a result of current moves to nearly double airplane production by having the auto industry build planes. The plan, now under discussion between the Defense Commission and the industry, is that parts be fabricated in existing auto factories, but that a group of huge new assembly plants be built by this government and the British where the planes can be put together. It is still too early for much to have been decided about these plants, but they will be located in the Middle West, near to but not in Detroit, and the cost will run into tens of millions. Another big project is a British plan for construction of a shipyard in this country of the Hog Island type to turn out large quantities of simple freighters.

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Commission to tackle Norfolk area pollution

A five-member commission, succeeding the present Hampton Roads Sewage Disposal Commission in Virginia, will soon be appointed by the governor of that state to tackle the pollution problem in the Norfolk area. The outgoing commission, of which Robert Preston of Norfolk was chairman, rendered its final report on Nov. 14.

Establishment of a sanitation district in the area on both sides of Hampton Roads was approved by voters on Nov. 5. It is estimated that sewage disposal plant construction will require $8,000,000 to $10,000,000.

Meantime Norfolk, which is opposed to the district plan, has undertaken studies of its own with a view towards building a sewage treatment works. At present a 250,000-gallon demonstration plant, using sand filters of a new type, is being operated.

A.A.E. seeks recognition for engineers in defense

Greater recognition for engineers on the defense program is sought by the American Association of Engineers. At a recent meeting of the board of directors of the association a resolution was adopted urging the "appointment of engineer experts to key positions as assistant to the secretaries of War and Navy to direct planning of projects in connection with national defense." The board also urged that all design and supervision of defense construction be done directly under engineers "rather than have contractors employ engineers for design and supervision, which reverses the usual and desirable relationship between engineer and contractor."

Appointment of a representative of "employed engineers" on the National Construction Advisory Committee was a further recommendation of the board "to prevent exploitation of the patriotism of engineers of this rank whose economic interests are not covered by federal labor legislation which extends to all other subordinates."

Extension of the classified civil service act to field employees of the U.S. government was endorsed by the board.

Photo by Allis-Chalmers Mfg. Co.

Bulldozer replaces spade in groundbreaking ceremony

In tune with the times, Guy W. Vaughan, president of the Curtiss-Wright Corp., used a diesel tractor and bulldozer instead of the traditional spade to break ground on Oct. 23 for the largest single industrial unit to be started in the national defense program, a $37,000,000 airplane engine plant at Lockland, Ohio. The main building, 1,064 x 1,380 ft., will cover nearly 35 acres. The foundry will take up 160,000 sq. ft., and office and engineering quarters will occupy 66,000 sq. ft.

Albert Kahn, Inc., Detroit, is architect on the project. Separate construction contracts have been let to several firms.

Riding the tractor in the above view are Brig. Gen. O. P. Nichols (left), Wright Field, Dayton; Emil Schram (center), chairman of the RFC, and Vaughan.

Comment and Discussion

Readers' opinions on matters that concern engineers and contractors

Aerodynamic Stability of Suspension Bridges

Sir: It is well known that most airplane wings become structurally unstable at a certain flying speed. The reason is that an accidental twisting of the wing produces aerodynamic forces and moments which tend to increase the accidental twist.

Consider now the truss of a suspension bridge. It is evident that if the truss oscillates laterally it becomes slightly twisted. Then the wind blowing perpendicular to the span produces lift forces. The center of pressure of those lift forces lies about half way between the "leading edge" (the edge exposed to the wind) and the center line of the truss, at the so-called quarter-chord point. The moment of the lift forces is resisted by the torsional stiffness of the truss and the reaction of the cables. Both the elastic restraining moment and the moment of the lift forces are proportional to the twist. However, the proportionality factor of the restraining moment is constant, whereas the aerodynamic moment increases with the square of the wind velocity. Hence there must exist a critical value of the wind velocity, for which the structure becomes unstable.

The effect of each cable suspension can be replaced approximately by the effect of an axial tension of the magnitude H applied to the truss in the spanwise direction, provided H is equal to the horizontal tension in the cable. It can be shown that this tension increases the torsional stiffness of the truss by the amount $Hd^2/2$, where d is the distance between the two cables.

The moment of the lift forces per unit length of the truss can be estimated by the equation

$$M = k\,b^2\,q\,\theta/4 \qquad (1)$$

where b is the width of the truss, q the dynamic pressure corresponding to the wind velocity, θ the twist angle in radians (angle of attack of the wind), and k a numerical factor of the order of magnitude 5.

Then we can write the following equation for the angle of twist θ as function of the coordinate x, taken along the span:

$$\left(C + \frac{Hd^2}{2}\right)\frac{d^2\theta}{dx^2} + k\,b^2\,q\,\theta/4 = 0 \qquad (2)$$

This equation is solved by a sinusoidal distribution of the twist angle, with $\theta = 0$ for $x = 0$ and $x = l$, i.e. at the towers. Then we find that the twist angle can increase indefinitely, i.e., the structure fails, if

$$k\,b^2\,q\,\theta/4 = \left(C + H\frac{d^2}{2}\right)\frac{\pi^2}{l^2}$$

or

$$q = \frac{4\pi^2}{k}\,\frac{\left(C + H\dfrac{d^2}{2}\right)}{b^2\,l^2} \qquad (3)$$

In many cases, for example probably in the case of the Tacoma bridge, the torsional stiffness C is small in comparison to $Hd^2/2$, and d and b are approximately equal. Then we have

$$q = \frac{2\pi^2}{k}\,\frac{H}{l^2} \text{ or about } 4\,\frac{H}{l^2} \qquad (4)$$

Substituting in Eq. (4) for H the expression $H = \dfrac{wl^2}{8f}$ where w is the dead weight per unit length carried by one cable, and f is the cable sag at midspan, we obtain

$$q = \frac{1}{2}\,\frac{w}{f} \qquad (5)$$

It is seen that for identical values of f the critical wind load is proportional to the weight of the structure per unit length and is therefore especially low for narrow bridges.

The main span of the Tacoma bridge is $l = 2{,}800$ ft.; with this value we obtain the following table for the critical wind velocity as function of the cable tension H. The exact value of H in the case of the Tacoma bridge is not known to the writer. (It was about 12,000,000 lb. per cable under dead load.—EDITOR.)

Cable tension in million lbs.	Dynamic pressure lb./ft.²	Critical wind velocity, mi./hr.
5	2.55	32
10	5.10	46
15	7.65	55
20	10.20	63

It must be remembered that these values are somewhat low because the torsional rigidity of the truss has been neglected. On the other hand the direct wind pressure on the lateral surface of the truss produces a tendency for lateral buckling which is unfavorable for the stability. Also, the horizontal wind force is increased by the twist, since the projected area increases.

Finally it must be noted that, as in the case of wing flutter, the structure usually fails before the wind velocity actually attains the theoretical critical value. Before this speed is reached the amplitude of the lateral oscillations increases to such extent that the structure breaks down. The critical section, for example in the case of lateral oscillations the section of maximum twisting moment, is probably at or near the ends of the span.

THEODORE VON KARMAN
Director, Daniel Guggenheim Aeronautical Laboratory, California Institute of Technology, Pasadena, Calif.

(Dr. von Kármán's computation as tabulated evidently applies to the fundamental oscillation, without node in the main span. As the motion of the bridge preceding its collapse showed unsymmetrical vibrations, and the motion which seemed to predominate had a node at midspan, Eq. (3) and following would be modified by changing π to $n\pi$, with $n = 2$ for the principal motion. In this case q would become 4 (H/l^2), and the value of q would be about 25 lb. per sq. ft. in place of about 6 lb. But the participation of side spans and towers, making the bridge "softer," would again decrease this figure substantially.—EDITOR)

Pile Bearing Increased by Close Driving

Sir: With reference to the article on p. 172 of the Aug. 1 issue by Mr. Lidicker, and the comment by E. H. Connor in the Sept. 12 issue on p. 324, it would appear that certain fundamentals in soil mechanics should be given consideration in any discussion of the results to be obtained in driving bearing piles in closely spaced clusters.

Driving tapered tubes into various kinds of material has provided an opportunity to study the behavior of various soils and their reactions on the tube, and likewise verify Timoshenko's research work having to do with collapsing pressures.

It is common knowledge that driving displacement type piles in substantially non-compressible material confined by sheetpiling will behave in one of two ways. The pile length will become shorter for a given resistance, or for a given penetration the individual pile resistance will be greater, and if the driving is carried far enough to overcome the ultimate shearing stress of the soil, upheaval may occur, partially destroying the bearing power of the individual piles rather than improving it. It is for that reason that care should be used in selecting a pile with the proper taper and displacement. This is best determined by the actual driving of test piles of different diameters, tapers and displacement.

On the other hand when you remove the confining medium—that is, the sheetpiling, and depend on the outer rows of piles in a cluster, experience shows readjustment takes place in the vectorial components of the forces around each pile. This tendency represents itself by a reduction in the value of the horizontal component of the forces supporting the pile, reducing the piles' bearing capacity.

The supporting power of the pile, being a combination of the total skin friction, which is a function of the coefficient of friction, the total pressure normal to the surface of the pile, and the reactive forces under the nose of the pile, will automatically improve with deeper penetration when piles are driven in uniform material.

This fact coupled with usually a lower cost per ton of carrying capacity, when driven deeper and to a higher resistance, tends toward a more economical and more reliable foundation—the latter case utilizing the effectiveness of surcharge and eliminating the loss of carrying capacity in certain soils by working on close centers.

Usually when the character of the soil is such that it can be consolidated, the consolidation is only temporary, dissipating itself with time; eventually the cluster acts as a unit with a total bearing capacity something less than the capacity represented by the number of piles times the unit bearing capacity for each pile in the pile cluster.

A. R. MILLER
Manager Piling Sales & Engineering
The Union Metal Manufacturing Co.
Canton, Ohio

Pile Bearing in Sand

Dear Sir: From the first paragraph of Mr. Connor's letter (*ENR*, Sept. 12, 1940, p. 324), it might be inferred that the results of the pile bearing tests, as reported in my article in the Aug. 1 issue, were unexpected. On the contrary the building up of resistance due to the compaction of sand was known to exist, but to my knowledge the amount had never been evaluated by test, particularly under such favorable circumstances as the loosely deposited uniform sand fill presented.

Considerable compaction of the sand due to the vibration of driving was anticipated, and a general settlement of the area was expected even though the volume of the timber piles themselves absorbed a substantial percentage of the volume decrease due to compaction. In *Civil Engineering* of February, 1939, E. J. Christenson presents the results of settlement observations caused by the piledriving, showing that the surface of the area lowered as much as 1 ft. with an average of about 0.6 ft.

The "flowing" of material which Mr. Connor speaks of due to the driving of piles will occur in silts and clays, but in a relatively clean and loosely deposited sand the decrease in volume is caused by the increase in density of the sand during the driving.

Driving individual piles at scattered locations has proven unsatisfactory as a method of determining pile lengths because of the variation in indicated bearing caused by various densities of sand. The purpose of the tests described was to evaluate the increase in supporting power of a pile due to compaction of the sand, and to select pile lengths from the results of driving a group of piles.

However, Mr. Connor reminds engineers that another important factor also to be considered is the supporting power of the material beneath the piles. It is well to emphasize this point as one which should be investigated in each foundation problem. Since it is generally impracticable to load test a large group of piles simultaneously, the bearing on the material below the pile tips must not exceed the allowable. Separate tests on undisturbed samples may sometimes be necessary to determine the maximum allowable.

In the case under discussion there is a minimum of 9 sq. ft. of bearing under each pile (and more if the spreading of the load is accounted for). Since the maximum load per pile is about 30 tons and the normal load 25 tons or less, the resultant bearing on the sand under the piles is only 3 or 4 tons per sq. ft., and it was felt that this was well within the allowable load. The behavior of the completed structure has verified this.

So far as a reasonably large group of piles is concerned, the perimeter of the group need not be equal to the sum of the pile perimeters. In such a case a substantial portion of the load would be transmitted from the overlapping bulbs of pressure into the sand below the pile tips.

W. Z. LIDICKER
Engineer, U. S. Engineer Office
St. Paul, Minn.

Rigid Connections

Sir: I have read with some interest the article entitled "Design Economy by Connection Restraint," by Prof. Bruce Johnston and Robert A. Hechtman (*ENR*, Oct. 10, 1940, p. 484). The authors have presented arguments for using the restraint of end connections in designing ordinary steel beams. They emphasize the element of economy associated with the use of the lighter beams which results when advantage is taken of the partial continuity existing at the ends of the span.

There is some danger in interpreting the authors' conclusions too broadly. For the type of wind-bracing, such as the angle connection illustrated in Fig. 6, or the split-I connection, one is justified in designing a beam on the basis of continuity. When an ordinary standard beam connection is used, however, one which is fastened only to the web at the end of the beam, the writer believes that it is not advisable to utilize the end restraint developed by such a connection in reducing the simple-span moment in the beam. The angle of rotation between the fixed-end condition and the simply supported condition is very small; and if one wishes to guard against the possibility of an overload, in other words, against the possibility of "What may happen rather than what usually happens," he will continue to design beams with such connections as simply supported beams.

HAROLD E. WESSMAN
Professor of Structural Engineering
New York University

EDITORIALS

VOLUME 125 NUMBER 21

November 21, 1940

Dynamic Wind Destruction

INCOMPETENCE OR NEGLECT cannot be blamed for the Tacoma Narrows collapse, greatest bridge failure in history. Acknowledged leaders in bridge engineering had built the structure, and both design and construction were carried out with conscientious care. The collapse resulted from actions and forces previously ignored or known to be unimportant that became dominant in the new range of magnitude and proportions of this exceptional bridge.

The element of new magnitude was involved also in the fall of the uncompleted Quebec cantilever 33 years ago; there, practices approved by experience in smaller dimensions proved to be faulty when applied to the huge 1,800-ft. span. But, aside from the careless haste that marked its record, the 1907 tragedy taught only lessons of detail, chiefly shortcomings in the construction of columns. The lesson of Tacoma is more fundamental. It tells us that dynamic action may outweigh static action and that a bridge amply strong to resist all probable static forces may be fatally weak under dynamic attack.

The fortunate presence on the bridge of Prof. F. B. Farquharson, who for months had been studying the structure in field and laboratory, makes the story of the event unusually complete. The bridge went into intense resonant oscillation under the cumulative effect of undamped rhythmic forces. Coupled with the wind tunnel findings, the writhings of the bridge point to dynamic actions quite like those which have wrecked many an airplane through wing flutter.

Any elastic structure can be thrown into resonant vibration or oscillation by small impulses repeated at the right intervals. Where wind is involved, a highly flexible structure is most vulnerable, because in its large range of motion it can absorb much energy from the successive wind impulses. The Tacoma Bridge was exceptionally flexible, and its thin, narrow ribbon of steel, only 8 ft. deep and 39 ft. wide in a length of more than half a mile, could and did undulate up and down like a thing of life.

On the day of the bridge collapse, moreover, this up-and-down undulation, apparently unaccompanied by any serious amount of lateral sway, changed to a twisting that rapidly grew to extreme degree. Whether the twisting was started by the lift of wind eddies or by slight elastic unbalance of the two sides is not important; once it was started, impulses of aerodynamic unbalance went into action to accelerate the motion.

The deck in fact was an airfoil, a wing section, and the wind currents sweeping across subjected it to rapidly varying lateral and vertical pressures —drag and lift, in aerodynamic terms—which were out of phase and thereby fed more energy into the structure than they took out. Dynamic twist and oscillation increased as more energy was fed in, until something had to give.

There was forewarning of this event in the behavior of three earlier bridges, where undulation also occurred. But in these—Thousand Islands, Deer Isle and Whitestone—it was almost negligibly small, and was successfully controlled by restraining devices; several years of service certify to the security of these structures. The occurrence of this new kind of trouble in long, light suspension bridges obviously flashes out a caution signal, which gives warning that bridge engineering cannot proceed on statics alone but must explore dynamic actions and aerodynamic effects.

Elastic models, wind tunnels and study of resonance and damping thus take their place in the bridge engineer's equipment. Whenever a highly flexible bridge is to be built, these tools will have to be called upon to prove its stability before construction begins. And until such proof is beyond challenge, extreme flexibility must be looked on with suspicion.

The Tacoma collapse is not an indictment of suspension bridges as a class. The scores and hundreds of such bridges that have stood firm and secure for years is full and ample defense. Nor are the principles of suspension bridge design placed under suspicion, for these bridges were proportioned by the same principles as those applied at Tacoma. But there is, as we now learn, a limit to the degree of flexibility compatible with due resistance to aerodynamic impulses, and this limit remains to be determined. It applies, however, to future bridges only, and does not cast doubt on the safety of those now in use.

Thorough investigation of the collapse is, of course, essential to establish the facts and phenomena with full precision. It is to be hoped that the federal government, which is interested because of the part played by the PWA and the RFC in

construction of the bridge, will participate or take the lead in such investigation to assure that it will be thorough and comprehensive. Bridge engineering advance as well as the public interest require that the teachings of the disaster be made clear beyond a shadow of doubt.

The Price of Speed

NEW YORK CITY HAS OPENED its first vehicle tunnel under the East River, a toll facility that will compete with the free bridges on the basis of giving faster service. Many vehicles are expected to use the new tunnel to avoid the long delays and dense traffic on the Queensboro Bridge, its chief competitor. The principal reason why the new tunnel can offer faster service is because of its elaborate approach and exit road system, especially in Manhattan, where two new north-south streets have been cut through six blocks to give many approach channels. At the Queens end, a long viaduct over railroad yards and local streets, gives traffic bound for outlying points on Long Island the quick getaway that is lacking at the Queensboro Bridge. Thus new emphasis is placed on the fact that the price that must be paid for a new crossing over a major obstruction to traffic is not the cost of the bridge or tunnel alone but must include expensive approaches as well if speed is to be attained.

Chicago Reaches the Goal

WITH THE ANNOUNCEMENT that work will begin on doubling the size of the world's largest sewage treatment works, the Sanitary District of Chicago makes an auspicious start on its second half century of service. Fifty-one years have elapsed since the district was created to develop a sanitary plan for Chicago and its suburbs and to build works for sewage disposal. Today the district can point to the approaching completion of its huge task of providing complete disposal for an area having a population of 4,700,000 and for industries whose wastes are equivalent to sewage from another 2,200,000 people. Chicago's progress in sewage disposal covers two eras. From 1900 to 1920 was the era of dilution, a period during which canals were built in order that Lake Michigan water could be used to flush sewage wastes away from the lake and into the watershed of the Mississippi River; to accomplish this some $125,000,000 was spent. Dilution methods proved to be inadequate and undesirable, and when the art of sewage treatment had advanced sufficiently, the second era began. From 1921 to now, $163,000,000 has been devoted to research and installation of treatment plants, and another $10,000,000 is to be spent for the new facilities at the Southwest Works. The work of the Sanitary District of Chicago has been a labor of cooperation by individuals rather than a proving ground for personalities. The results achieved during the past half century are a monument to this united effort, clear vision and devotion to a public responsibility.

New Housing Techniques

THOUGH THE DEFENSE housing program is comparatively small compared to that for subsidized public housing of recent years, there are already indications that it may prove more significant for the future of house building than the bigger program ever did. Sheltered under the wing of "defense", Coordinator Palmer, Administrator Carmody and Commissioner Reynolds are less exposed to adverse criticism than was the embattled USHA. It is perhaps for this reason that they show more inclination to experiment with new techniques and procedures. The peacetime housers experimented with financing; they nearly got housing accepted as a normal government function; and they managed to reduce public housing costs materially. But of technical innovation there was almost none from the $650,000,000 laboratory of the Housing Authority. The new program looks as if it would be different. Technological radicals have been crying for years that the mass-produced prefabricated house would solve the housing problem. Now it is planned to give test orders to the prefabricators, and there is prospect of orders big enough really to prove or disprove their theories. Similarly, the program as now laid out should demonstrate whether economies can be achieved through bypassing the present small-scale local distribution of house materials. If this same spirit is maintained, house building may prove to be the beneficiary of one of those incidental advantages that accompany the wastefulness of war.

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Model Tests Showed Aerodynamic Instability of Tacoma Narrows Bridge

N. A. Bowers
Pacific Coast Editor, Engineering News-Record

Undulation of 2,800-ft. span, a matter of concern from the beginning, led to installing brakes and tiedown cables —Tests of elastic model to develop effective means of control were followed by discovery of aerodynamic instability of deck through wind-tunnel tests—Fairing or shielding under consideration at time of collapse.

WRECKING of the 2,800-ft. highway suspension span over Tacoma Narrows near Tacoma, Wash., by cumulative undulation and twisting on Nov. 7 was the final chapter in a story of alarming unsteadiness and attempts to find means of preventing or controlling the snakelike movements of the bridge.

Even before construction of the steel towers was completed rumors of small undulations of the Whitestone Bridge at New York had led the engineers to decide on elaborate tests on an elastic model of the Tacoma structure. These tests, carried on by Prof. F. B. Farquharson of the University of Washington during the past summer, showed satisfactory control to be difficult.

Finally, within the past two months, wind-tunnel tests on models of the deck revealed conditions of aerodynamic instability believed sufficient to set up oscillations of the kind that proved destructive, and plans had been made for streamlining (fairing) or else shielding the sides of the stiffening girders. Collapse came while a decision was pending.

Details of failure

Major elements of the structure are shown by the drawings on the accompanying plate (see next page). As shown, the 17-in. cables held up a solidly constructed trough-shaped assemblage of stiffening girders and floor by means of suspenders spaced 50 ft. longitudinally, each comprising four parts of 1¼ in. 7x7 wire rope. The deck weighed about 110 lb. per sq. ft. The suspenders, of average test strength 165,000 lb. per rope with socket, had a factor of safety of about 6½ under dead load. Laterals were arranged in K-system, and were designed to resist in conjunction with the cables a wind pressure of 620 lb. per lineal foot of bridge. As a matter of fact the bridge proved quite stiff laterally. Side sway has been an almost negligible item in its movements, and even in the violent rolling on the day of the collapse a sight along the stiffening girders indicated that transverse sway in the main span did not exceed about 2 ft., less than one-tenth what the bridge could stand.

It is not known whether failure began in the suspender cables or in the girders or floor. A decided disalignment of the stiffening girder at midspan appears in a picture taken at a time when the concrete roadway had not yet begun to fall. Another picture, taken soon after the first concrete fell, shows a panel of lateral bracing broken loose and hanging down below the floor. The suspender connections apparently did not fail, as breaks occurred consistently in the ropes themselves. Frayed stranded ends swinging in the wind show the breaks occurred in different locations, and such examination as has been possible indicates that only one of the many broken suspenders has the socket still attached.

The main cables show no damage that can be seen from a distance, except that one of the diagonal braces put in recently at midspan between cable and girder flange to check longitudinal motion of the deck pulled the middle cable band along the cable and cut into it somewhat. Numerous broken wire ends appear to be protruding—either the wire wrapping or the wires of the main cable itself.

There is an 18-in. transverse kink in one stiffening girder of the Tacoma side span. But despite the 30-ft. sag of this span its 5½-in. concrete roadway slab is still intact except where broken by impact at the transverse joints.

Vertical wave motions of the bridge, observed ever since the floor system was completed, were not proportional to wind velocity. Sometimes very considerable vertical waves were produced by winds as low as 4 mph. and again there would be no waves in winds as high as 32 mph. Until Nov. 7 they had always been equal on opposite sides of the structure. They caused no damage so far as known, but they were disturbing to traffic and it was decided to try to control them. Different expedients were studied with the aid of the 1:100 dynamic model at the University of Washington.

Tie-down cables with stress-limiting links were put in the side-spans 300 ft. out from each anchorage, and at mid-span diagonal stay members were placed between main cables and girders. Hydraulic brakes to check longitudinal motion of the deck were installed at the towers. Consideration was given to placing diagonal cables between tower tops and deck. These expedients aimed at restraining the movement and steadying the bridge.

Aerodynamic instability

Then, about five weeks before the failure, attention was turned to the question of preventing wave motion before it could build up. On the chance that an aerodynamic cause might be found, a 1:20 model of the deck was built to represent a 160-ft. length of the bridge (model 2x8 ft.) and was tested in the wind tunnel. These tests showed the model to be aerodynamically unstable in certain winds, and this condition was concluded to be the source of the oscillations of the bridge itself.

The characteristic lift curve for the model representing the floor system as built, A in Fig. 3, has a reversal or negative angle which indicates instability. In this curve, as in those for modified forms of the model, the lift coefficient C_L is plotted vertically and the angle of attack of the wind

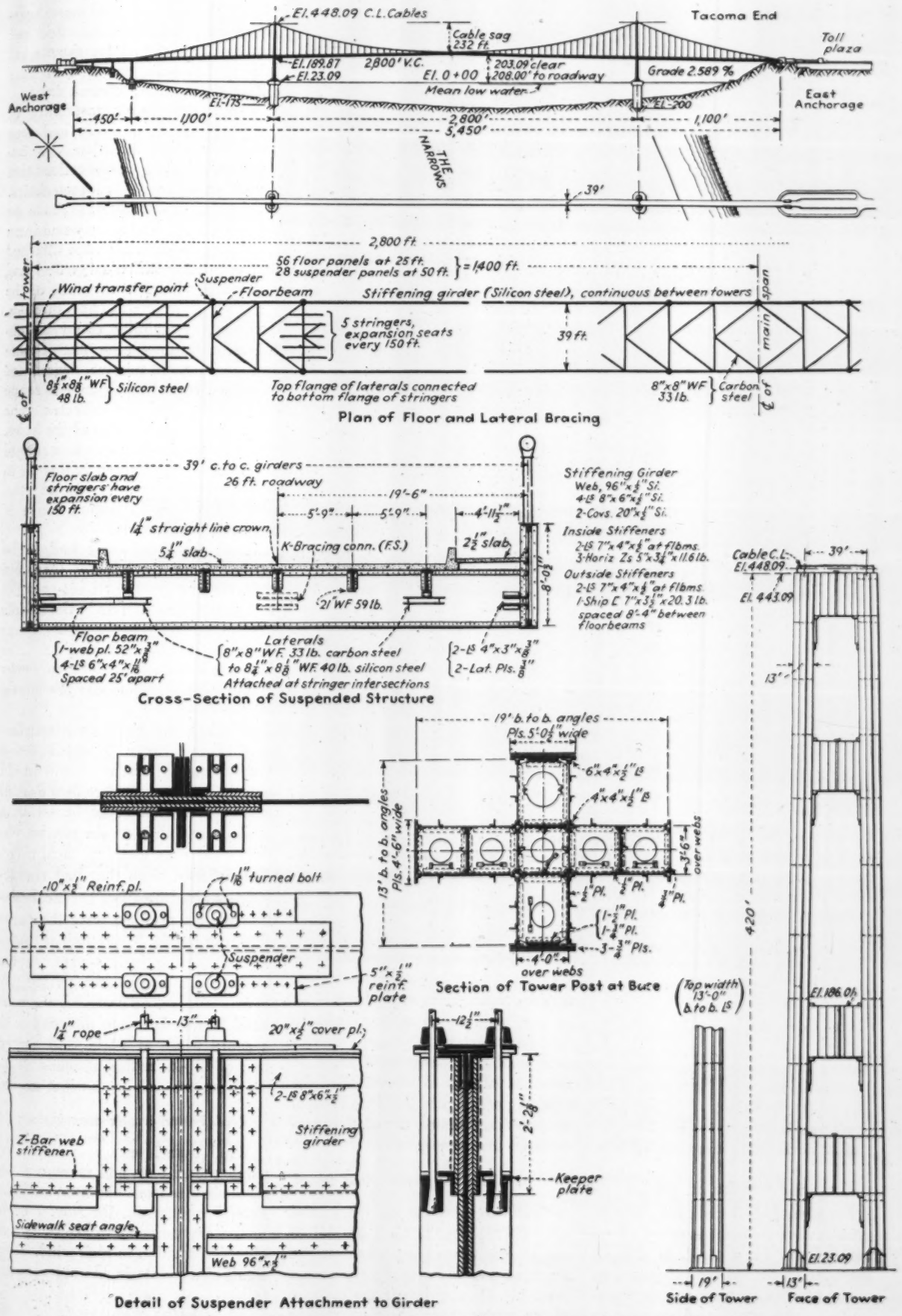


Fig. 1. Main dimensions and important details of Tacoma Narrows Bridge.

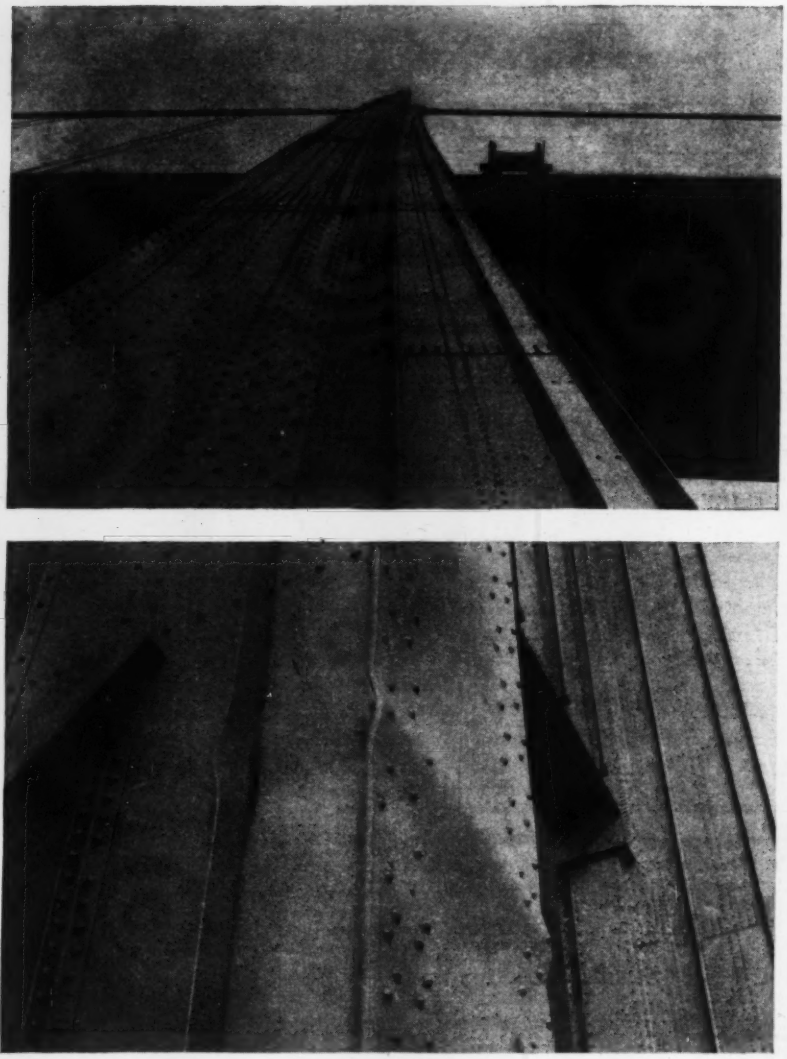


Fig. 2. (Top) When the deck of the main span fell, the steel towers of Tacoma Narrows bridge were bent back 12 ft. by the unbalanced pull of the side-span cables. (Bottom) Buckled plates and angles near the base of the tower tell of excessive bending stress under the severe deflection of the top.

horizontally. The lift co-efficient is related to total lift as follows:

$$L = C_L \frac{r}{2} A V^2$$

where

L = lift, lb.
C_L = lift coefficient
r = air density
A = horizontal area of surface, sq. ft.
V = wind velocity, ft. per sec.

For a wind of 45 mph. this formula gives an insignificant loading, of no consequence in a rigid as distinct from the suspended type of structure.

Danger lies in aerodynamic instability, which was found to exist in this structure only with wind in or very close to a horizontal direction, namely, through the angle from $-2\frac{1}{2}$ to $+4\frac{1}{2}$ deg. In an aerodynamically unstable structure the change in vertical wind component with angle of attack is subject to reversal and the resulting action may build up vertical wave motion whose cumulative possibilities are very great.

Fairing, shields and perforations tested

Wind tunnel tests on the model were all made with wind normal to the bridge centerline; other angles were not possible with the equipment available. On the day of the collapse of the bridge there was a quartering wind. This may have had some bearing on the unusually severe effect and also on the fact that the same wind velocity has been observed to have different effects on different days.

When aerodynamic instability of the floor system, as built, had thus been discovered, attention was immediately turned to modifications that would improve its characteristics. Changes of two kinds were made on the model: (1) the use of some form of fairing or deflector vane attached in front of the stiffening girders and (2) making holes in the webs of the girders to allow wind to pass through. The desired stability could be obtained, it was found, in either of these two ways, with relative degrees of effectiveness according to the shape and position of the deflector vane and the size and location of the holes. The scope of the tests on vanes ranged from deflectors of small radius up to a full fairing, presumably of steel plates, from top to bottom flange of the stiffening girders.

Small wind deflectors had to be farther from the girders than did those of a larger size. Holes near the top, near the bottom or in the middle of the girders were not nearly as effective as a combination of holes both top and bottom. Relative merits of different arrangements are shown in the lift-coefficient curves Fig. 3.

Although the holes gave satisfactory characteristics it was believed that they would cost more and would mar the appearance more than a small curved plate that could be set in a position where it would not be visible from cars crossing the bridge. The deflector vane that had characteristics best suited to a practical solution of this problem gave the curve shown in D, Fig. 3; preliminary steps toward putting such a vane on the bridge were being taken on the day of the collapse. As designed, the vane itself was to be made of wood with a chord width of 19 in. and curved face of $12\frac{1}{2}$-in. radius, attached 80 in. out from the web by bracket angles.

Prof. Farquharson's observations

Wind tunnel tests as well as dynamic model tests and measurements on the bridge itself had been made by Prof. Farquharson. On Nov. 7 early in the day he went to the bridge to observe its action in high wind. Some significant points in his observations follow.

At first, the wave motion of the

bridge had the frequency of 36 cycles per minute, often observed before. Some eight or ten wave frequencies had been observed, among which 12 cycles per minute had occurred perhaps more often than others; this is also the frequency of the towers, as determined by the U. S. Coast and Geodetic Survey after an earthquake a year ago. At about 10 a.m. the 36-cycle wave changed suddenly to one of 14 cycles, with a single node at mid-span. Then for the first time the vertical wave motion on the two sides of the bridge got out of step, causing a twisting in the floor system as one side went up while the other went down. This twisting was confined to the main span and did not pass into the side spans, Prof. Farquharson observed.

As the twist approached its maximum the deck tilted from side to side through vertical angles of more than 45 deg. with the horizontal; looking along the bridge lengthwise, lighting standards on opposite sides of the deck and at opposite wave phases were observed to cross at right angles. Motion on the far end of the bridge appeared to be greater than on the Tacoma end, and this is borne out by the fact that numerous light standards on the far side were snapped off although none went down on the Tacoma end until the final collapse. There were times when it seemed that the roadway would roll completely over, he said.

At the time of maximum violence the wave motion had a period of 12 cycles per minute. The acceleration exceeded gravity, as he felt the deck to drop out from underfoot and an automobile left on the span repeatedly shifted its position.

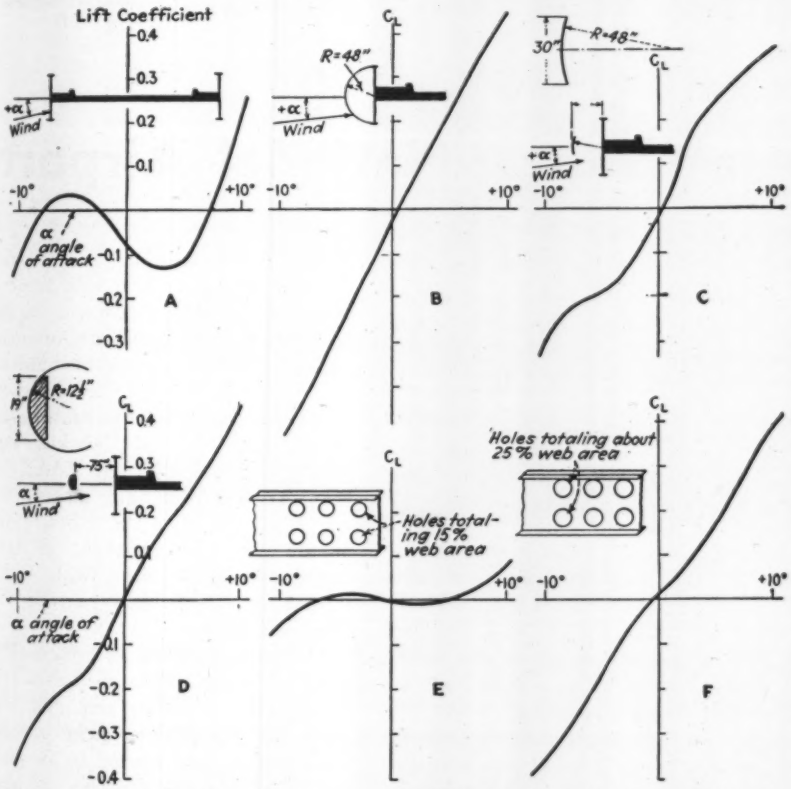


Fig. 3. Aerodynamic instability of the bridge deck was revealed by wind-tunnel tests on a model. The six curves show lift coefficient, plotted on angle of attack of the wind, for the deck as built and five modifications. Downward slope of the lift curve spells danger.

A. Deck as built. B. Deck with semicircular fairing. C. Deck shielded by 30-in. vane. D. Deck shielded by 19-in. vane. E. Girder perforated. F. Larger perforations.

Note that angles are positive when wind acts at an upward angle to the deck, or when in horizontal wind the leading edge of the deck is raised. Angles are negative when the wind blows from above the horizontal, or when in horizontal wind the leading edge of the deck is depressed.

After the stiffening girders buckled on both sides, a series of suspenders were snapped and broken ends were seen to fly high above the main cables. Failure of the floor system progressed rapidly, whole sections, some of them several hundred feet long, falling out at one time.

After the first large sections fell, the violent torsional movement almost ceased for a few moments, but quickly was resumed and this time the waves built up in the side spans, which had sagged 30 ft. as the main span cables rose. Shocks that then occurred in the side span, as successive sections of the main span dropped, were sufficient to twice throw Prof. Farquharson violently to the deck.

Later observations on towers

Detailed inspections of towers and cables and decision as to whether they must be taken down is not to be made until the decks have been removed from the side spans to relieve unbalanced loading. As this proceeds the expected return of the tower tops toward their normal position is taking place.

Examination of tower bases shows a separation of almost ½ in., at the extreme tension edge, between base plate and tower. This appears to have occurred after slippage in rivets that connect the tower with the steel anchor members going down into the pier concrete. These members bear indications of stretching and the concrete of the pier tops is damaged, apparently from upward pull on the anchorages. In addition to the bend in the towers and the buckled plates near the base, already noted, "ripples" can be seen on the compression face of the tower.

Salvage and reconstruction

The loss incurred in the collapse was covered by insurance, underwritten by more than a score of companies, which includes a use-and-occupancy clause to provide for payment of bond interest while the structure is out of service. It is thought that the insurance payment may be sufficient to cover the cost of reconstruction, though this cannot be known definitely until the remaining parts of the structure have been fully examined, and plans for the new bridge worked out. Even now, however, it appears probable that little can be salvaged from the floor systems of the side spans except possibly stringers and floor beams.

Courtesy Civil Aeronautics Authority

Fig. 1. The new airport at the nation's capital in October. The relocated Mt. Vernon Highway is at left.

Runways for Washington National Airport

***Contents in Brief**—Sand and gravel material dredged in to make the runways for the National Airport at Washington, D. C., is being stabilized by use of a small amount of soil binder and a large amount of mixing to make a cementless soil-concrete stabilized base 9 in. thick. A 2-in. first course and a 1½-in. top course of hot-mixed, hot-laid asphaltic concrete are made of the same dredged material, using softer-than-usual asphalts mixed at lower-than-usual temperatures to prevent cracking and undue oxidation and insure long service life where the kneading effect of traffic, depended on for conditioning of highway and street pavements, is absent.*

PLANNED FOR COMMERCIAL USE, the Washington National Airport, 3½ miles and 10 min. from the downtown postoffice, is approaching completion on a swamp site reclaimed by hydraulic fill (*ENR*, Aug. 31, 1939, p. 262). Stabilized base, approaching the characteristics of lean concrete, is built for the runways by addition of a sandy clay soil binder from the upland area to the sand and gravel pumped in, using the heaviest equipment available to mix and compact the top 12 in. of material into a 9-in. layer having a minimum weight of 130 lb. per cu. ft. The paving is a nominal 2-in. first course, which brings the surface to exact grade, and a 1½-in. final course, both of which are constructed with aggregates from the hydraulic fill operation, mixed and laid hot. To compensate for lack of the kneading action of traffic, soft asphalts are used having a penetration of 120–150 and are mixed within a temperature range of 225–240 deg. F. as this procedure preserves ductility and increases the life of the paving.

Design and construction of the airport is handled by a group of specialists assembled as a subcommittee under the Civil Aeronautics Authority and known as the Interdepartmental Engineering Commission. It is composed of representatives of the CAA, Corps of Engineers, Public Buildings Administration, Public Works Administration and Works Projects Administration with the cooperation of other federal agencies including the Public Roads Administration. The CAA made the runway layout; the U. S. Engineers designed and are constructing the runways in accord with methods worked out jointly with the PRA and CAA; the PRA handles tests and inspection; and WPA supplies the labor. Asphaltic concrete pavement is laid under private contract, with WPA furnishing a stipulated part of the unskilled labor.

The airport covers an area of 729 acres of which about one-quarter is reserved for building area and 556 acres for the landing field proper. Construction was started in November, 1938, and the runways are now usable with access roads far enough along to permit emergency use soon. The hydraulic work required moving 18,479,000 yd. of material, supplemented by more than 2,000,000 yd. of dry earth from the upland area of the field. The terminal building, heating plant, sewage and fire pump house and some of the hangars are now under construction, with com-

pletion expected in early spring.

Dredged fill was placed directly in the runway areas to El. +20, 5 ft. above final grade, with the expectation that the excess would be bulldozed off to make a landing strip 500 ft. wide capable of supporting a plane or ground equipment. Pending drainage and solidification of the lighter, soft material deposited between dredge lines, this excess fill has been pushed far enough to give at least 300-ft. width for each runway. The surcharge material expedited settlement and permitted removal of enough of the excess to furnish a solid base as well as to provide stabilization material for the taxiways, aprons and the airport roads.

A special study of the materials available resulted in a design to produce a stabilized, compacted sand and gravel base by addition of a small percentage of sandy clay soil, selected from the several types available in the upland area which required excavation. Determination of the amount of soil admixture required for proper stabilization was made by an analysis of the sand-gravel material existing in the grade for areas not exceeding 500 sq. yd. and deficiency of fines made up by addition of sandy clay soil binder in the proper percentage to meet one of the permissible gradations of the master chart, shown with a typical sample analysis in the accompanying table. After the percentage of binder required is determined, the longitudinal length a 2-cu.yd. transportable bucket will cover is computed for a strip equal to one-eighth of the runway width. Binder material is brought in and dumped for spreading by hand labor.

After the fill is graded to approximate elevation it is loosened to a depth of about 12 in. by use of bulldozers, patrol blades, and cultivators with a ripper used when required. Mud pockets and other unsuitable material deposits are

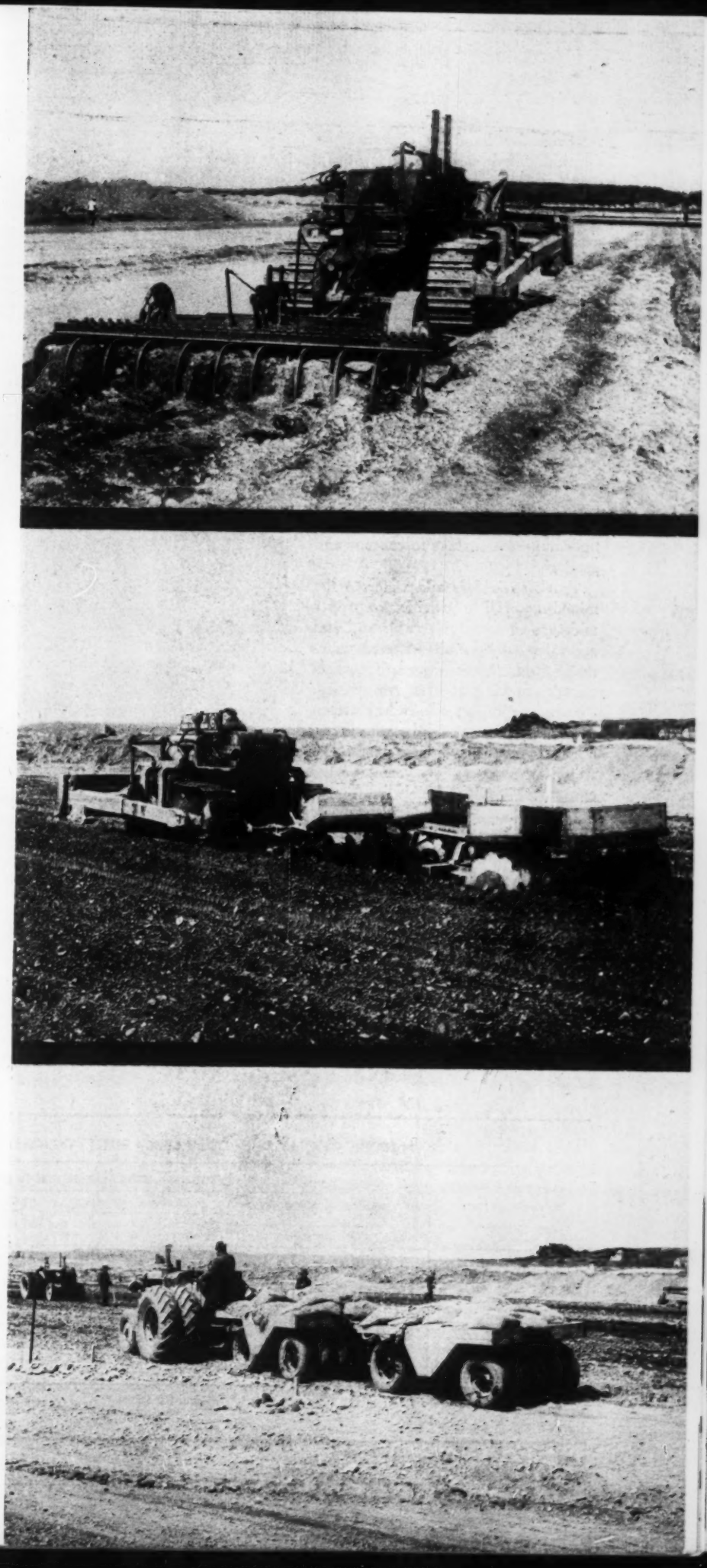


Fig. 2. Three steps in the preparation of runway subgrade. (Top) Heaviest cultivator available mixes soil to loose depth of 12 in. (Middle) Two-way tandem disc, 28-in. blades, is weighted to force full 12-in. penetration for mixing. (Bottom) Pulled by a wheel tractor, pneumatic-tired rollers compact properly mixed and moistened material.

removed, by hand or clamshell as the individual case warrants, to a depth of 3 ft. for small pockets and deeper for large ones. Tractor-drawn cultivators work the sand-gravel fill material into a loose homogeneous mixture from which hand labor removes all stone larger than a man's fist. Binder soil is then added by hand spreading over the measured and staked area and is plowed into the loose material by tractor-drawn four-bottom gang plows working to a depth of 12 in. To secure thorough mixing the cultivators are again used, followed by 28-in.-diameter tandem discs. Following the mixing patrol blades grade the surface of the loose mixed material to the shape required to bring the compacted surface to contour and grade.

Truck sprinklers add water to the mixed material to provide the proper moisture content for compaction and are followed by multi-wheeled pneumatic-tired rollers drawn by wheel tractors making at least two passes. The rollers impart a kneading action in the compaction, as the wheels are eccentrically mounted to produce a wobbling motion; job name for the equipment is "wobblies." Motor patrols keep the surface properly shaped during the initial rolling, which is followed by 10-ton 3-wheeled rollers and lighter tandem rollers for final compaction and smoothing.

Following the "wobblies" but ahead of the conventional rollers, tests are run to check the density and load bearing qualities of the compacted surface. Subsurface pockets of unsatisfactory material are located by operating a 12 to 14-ton solid-tired truck over the suspected areas.

Fig. 3. Mud pockets under stabilized base were removed by hand methods.

Fig. 4. Proper proportioning, adequate mixing and heavy compaction give a texture to the soil resembling lean concrete. Material at top has been primed.

For the relocated Mt. Vernon Highway, which cut across the site, and other areas of dry fill such as taxiways, aprons, and parking areas, the base is built in two 6-in. layers by hauling in sand and gravel from storage piles produced by the dredging operation. Soil binder is added and mixed in by discs and patrol graders, and each layer compacted with the rollers.

A tar prime coat is applied to the completed base, (except under concrete pavement) by a pressure bituminous distributor at an average of ¼ gal. per sq.yd. but varying somewhat due to differing absorption of the base.

In general, stabilized base has been constructed at a rate of about 20,000 sq.yd. per week but operations are often scattered and diversified so that exact amounts completed vary.

Paving

Design of the paving is based on expectation that the prepared base will remain firm under all weather and climatic conditions and provide full support to the surfacing to carry the load of a 65,000-lb. transport plane plus 50 per cent impact loading. Actual load tests have established bearing values uniformly in excess of 85 lb. per sq.in. (6 tons per sq.ft.) on the compacted base course. Tests indicate that even under the most extreme conditions, short of crash landing, the unit load on the paving does not increase appreciably, as the tire print area increases almost in direct proportion to the impact load even up to 200 per cent. Obviously the unit load cannot greatly exceed the unit air pressure in the tires, generally held at around 65 lb. Therefore, increased weight of flying equipment should not appreciably affect the unit loading on the runways as it is unlikely that tire pres-

MASTER GRADATIONS COMPARED WITH TYPICAL FIELD ANALYSES

Sieve Designation	Master Gradation: Percentage by weight passing square mesh sieves — A.A.S.H.O. T-27			126+00 — Field Analysis, E-W Runway				Field Analysis, Parking Area			
				Retained		Passing per cent	Variation from grading limits Type C	Retained		Passing per cent	Variation from grading limits Type —
	Grading B	Grading C	Grading D	Weight, grams	Per cent			Weight, grams	Per cent		
3 in.	100	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
2 in.	65–100	100	.....	.....	.....	.....	.....	.....	.....	.....	.....
1½ in.	.....	70–100	.....	.....	0	100	.....	.....	.....	.....	.....
1 in.	45– 75	55– 85	100	250	15	85	.....	.....	.....	.....	.....
¾ in.	.....	50– 80	70–100	341	21	79	.....	.....	.....	.....	.....
⅜ in.	30– 60	40– 70	50– 80	623	38	62	.....	.....	.....	.....	.....
No. 4	25– 50	30– 60	35– 65	883	54	46	.....	.....	.....	.....	.....
No. 10	20– 40	20– 50	25– 50	1,070	65	35	.....	0	0	100	.....
No. 40	10– 25	10– 30	15– 30	1,386	84	16	.....	3	6	94	.....
No. 200	3– 10	5– 15	5– 15	1,596	97	3	−2	123	25	75	.....
				Passing No. 200 sieve.....50 grams Wt. of total sample.....1,646 grams				Passing No. 200 sieve.....373 grams Wt. of total sample.....496 grams			

sure will ever exceed 75 to 85 lb., and the larger tire area distributes the pressure.

Soft asphalt

The kneading action of traffic which keeps the asphalts used in street and highway paving mixtures dense and watertight cannot be expected on airport runways, hence it is necessary to compensate for lack of travel by use of softer asphalts and richer mixtures prepared at the lowest practicable temperatures to prevent cracking and undue oxidization. The turnarounds at the ends of the runways get some abuse and are specified to be of 85–100 penetration asphalt to resist displacement due to twisting action in braking and turning. Material ranges are generally 5.3 to 5.5 per cent of asphalt by weight for the base course and 5.7 to 5.9 per cent for the surface course and have a penetration of 120–150, with a drop after recovery of less than 5 per cent due to the low mixing temperatures, 225 to 275 deg.F. with 240 deg.F. about the average. Temperatures of materials in the elevator for hot aggregates, the asphalt storage tanks and the weigh hopper are visually recorded by an electric pyrometer located at a convenient point away from the plant. The density of the compacted base and surface courses ranges from 94 to 96 per cent of the theoretical maximum.

Runway layout

The N-S and the N.W.-S.E. runways are each paved 200 ft. wide; the N.E.-S.W. and E.-W. ones 150 ft. wide and the taxiways 75 ft. wide. All have a 1 per cent straight slope from the center to the edge for drainage. Parabolic or circular sections are not being used as they have a relatively flat surface in the center which does not shed water rapidly and which would require impractical accuracy to prevent ponding water.

Total runway paving, including aprons, is 677,000 sq.yd. Paving over the stabilized base requires about 120,000 tons of asphaltic concrete, contracted for on a ton and gallon of ingredients basis to permit flexibility in the amount, proportions and location of its use.

The contract called for 80,000 tons to be placed between May 3, 1940, and Sept. 11, 1940, mainly in the runways and taxiways of the field proper and in a second construction period ending Nov. 30, to place up to 60,000 additional tons, as and where required. The supplemental requirement is now estimated at 40,000 tons and the locations have somewhat overlapped due to the decision to rush the highway relocation to permit removal of the existing road.

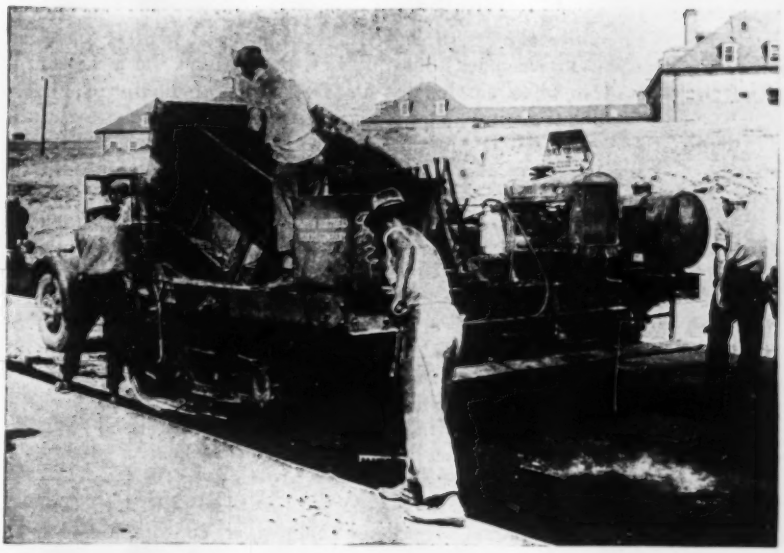


Fig. 5. On relocated Mt. Vernon Highway self-propelled spreader and strikeoff lays single course hot mixture on 9-in. concrete base.

Contractors' plant

Aggregate for paving is furnished the contractor from sand and gravel stocked near the plant location during the dredging operation. Unclassified material from the stockpile is taken by carrying scraper to a grizzly, then up a bucket elevator to the top of a processing plant where it is segregated into classifications passing ¼, ¾, 1¼ and 2½-in.-square openings. Material passing the 2½-in. and retained on the 1¼in. is crushed for additional aggregate, but the larger material is used for other purposes. A small portion of the excess dust from the fines is used for the base stabilization but most of it is wasted. The aggregates are drawn from the storage bins onto vibratory feeders, (which are set to proportion the materials automatically by varying the pulsations of the vibrators) and passed by them to the drier of the plant. Liquid asphalt is trucked in from Baltimore with storage provided in two 10,000-gal. tanks located near the plant.

Theoretically capacity of the asphalt plant is about 120 tons per hr. for the 5,000-lb. batch twin pug mill type mixer employed. The contract requirement is a 75-ton per hr. plant. Driers are capable of supplying the same quantity of material but the best continuous full plant operation has seldom exceeded 100 tons per hour. At the top of the hot elevator another vibratory screen seperates the dried material into gradations passing the 1½-, ¾, ⅜-in. and No. 8 screens. Banked hand levers operate gates for proportioning into a cumulative dial scale weigh box. Mineral filler and asphalt are also proportioned by weight. Required mixing time is 15 sec. dry and 45 sec. after the asphalt is added, both controlled by an electric timing device.

A contract offer of the airport authorities to furnish electric power at the plant for a stipulated installation and use charge was not accepted, the contractor electing to use diesel-operated power units for all requirements. One of the five major power units drives a direct-connected generator supplying lights at the plant and power for several motor drives at isolated points.

Average haul from central plant to the point of placement is slightly more than ½ mile except for the relocated section of the Mt. Vernon highway which is paved with a single 2-in. course over a 9-in. concrete base. Extra compensation is provided in the contract for any hauls over 1½ miles, which includes travel around detours, as usable roads are furnished the contractor. Hauling is

by truck taking either two or three 5,000-lb. batches. Trucks back into the front of the four-wheel combination spreader and finsher and dump as the machine pushes the truck ahead. A manila rope holds the truck at the proper discharge point, preventing dumping ahead of the spreader and tearing up of the prime by truck braking. Two pavers are in use at one time, traveling in echelon, each laying and partially compacting parallel strips 10 ft. wide.

Rolling is done by tandem and three-axle rollers, the latter being used ordinarily only on the wearing course. Compaction at 10-ft. strip joints is obtained by rolling only up to a foot from the edge until the next strip is placed. Diagonal and cross rolling is permitted to secure the required surface but is not often used despite the close tolerance of ¼ in. of established grade for base and wearing course and ½-in. variation in 10 ft. for smoothness of each course.

Drainage of the field is primarily to remove surface water on the runways quickly and consists of easy slopes from the runway edges which carry the water to drop inlets. From these the flow is directed through concrete pipe, parallel to and 50 ft. beyond the runway pavement, to the river, where flap valve outlets protect the drainage system against the backflow of a river flood. No underdrainage of the runways is provided as the sand-gravel material is expected to be self-draining. However, specially designed open joints in lateral pipe lines will assist to care for free water from any source.

Costs are moderate

Stabilization of the base to a depth of 9 in. has cost about 50c. per sq.yd., of which 30c. represents the cost of rental and operation of equipment and direct overhead and the remaining 20c. the cost of WPA labor. Asphaltic concrete pavement has cost about 60c. per sq.yd., exclusive of engineering and unprocessed aggregates, of which 12c. goes for WPA labor assigned the contractor. Contract bid was 5¼c. per gal. for asphaltic cement, and $4.25 per ton for limestone filler plus $1.07 per ton for mixing, placing, and rolling the asphaltic concrete.

H. H. Houk is resident engineer for the CAA which has had responsibility for design and general supervision of all aeronautical facilities. Maj. J. B. Newman, Jr. is in charge for the Corps of Engineers, U. S. Army, who have made the hydraulic fill and directed the surfacing work. Also for the Corps of Engineers E. J. Merrick, Jr. is chief of the engineering division, Gayle McFadden is chief of the operations division, and B. J. Bell is paving engineer. For PRA (whose general offices and laboratories on the airport site will be taken over soon by the airport and CAA personnel) E. L. Tarwater has general charge of the inspection of the runway and road construction. The relocated section of the Mt. Vernon Highway was designed by PRA and built by the Corps of Engineers with a maximum percentage of WPA labor. For Warren Bros. Roads Co., H. W. Makemson is superintendent and E. M. Pearson plant superintendent.

An All-Time Peak Pumpage

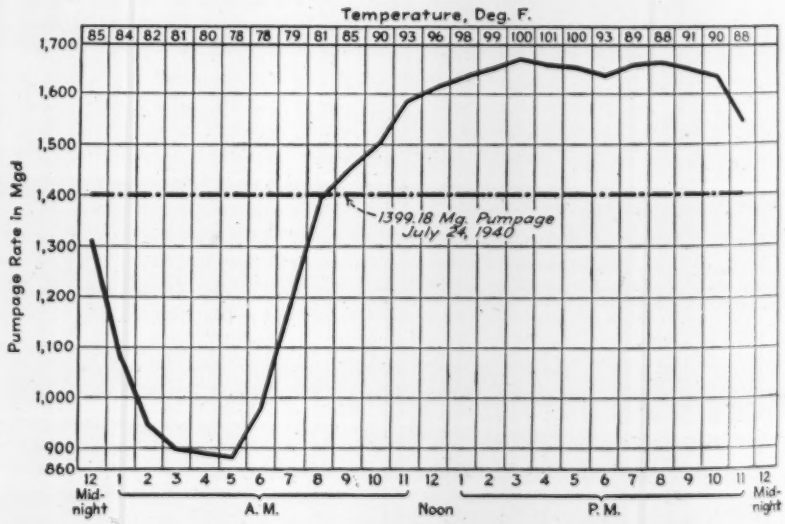
A HOT DAY was July 24 in Chicago and the water consumption hit an all-time peak, just under 1,400,000,000 gal. for the 24-hr period. The rate during the hour of maximum demand was 1,660 mgd. L. D. Gayton, city engineer, has plotted the hour-by-hour changes as shown in the accompanying graph.

On the previous day, July 23, a former record of 1,365 mgd was broken by a pumpage of 1,367 mgd. Pressures at the pumping stations ranged from 18 to 59 lb. per sq. in. with most stations maintaining a pressure above 30 lb.

Temperature seems to have a definite bearing on consumption demands, the peak pumpage of 1,661 mgd coming when the mercury had risen to 100 deg. F. During the next hour the temperature rose another point, then receded to the 100 mark. Strangely, the pumpage dropped to 1,649 and shortly to 1,645 mgd.

On July 25 an unusual pressure rise occurred in the system because of a change in wind direction and intensity together with rapid variations in barometer pressures and violent changes in lake levels. From 8:30 p.m. to 9 p.m. the 96-deg. F. reading dropped to 75 deg. A hot wind from the south at moderate velocity changed suddenly to a cold 44-mph wind from the north and this resulted in rainfall. Lake levels receded from 9 in. above datum to 18 in. below datum.

The demand for water dropped suddenly, creating a rapid rise in pressures before the pumpage rate could be reduced. Average rises were 10 lb. but in the northwest side increases were 16 lb. and in the south side 18 lb. This was attributed to the cessation of lawnsprinkling when the rain came.



Hourly pumpage rates in Chicago during high temperature period of July, 1940.

Timber Facing Prevents Dam Leakage

Paul Baumann

Senior Assistant Chief Engineer, Los Angeles County Flood Control District

OBSERVATIONS of the rate of leakage through San Gabriel Dam No. 2, a rockfill 265 ft. high built by the Los Angeles County Flood Control District, have shown that the facing of laminated timber planks put on this dam several years ago has justified the unusual design for a waterproof membrane. Construction of the dam was described in Engineering News-Record March 7, 1935, p. 343, and a description of placement of the timber facing appeared June 13, 1935, p. 836.

The concrete facing put on when the dam was built cracked and broke, as the dam settled, to such an extent that replacement or construction of a new impervious membrane was necessary. Accordingly, a facing consisting of three layers of 2-in. planking with tongue and groove connections on the ends as well as along the edges, was laid on an area of 156,000 sq. ft. This job, completed in April 1935, used 920,000 fbm of untreated lumber which cost $26 per thousand fbm and laid, under contract, at $33.50 per thousand fbm.

Under partial water load in the winter of 1935-36 (109-ft. head above No. 1 valve sill) leakage was 43 cfs. In the following winter under a head of 208 ft., maximum leakage was 124 cfs. Extensive repairs to the so called floater slab (which lies outside the timber facing) and also to the timber facing itself were made during low water in 1936 and 1937.

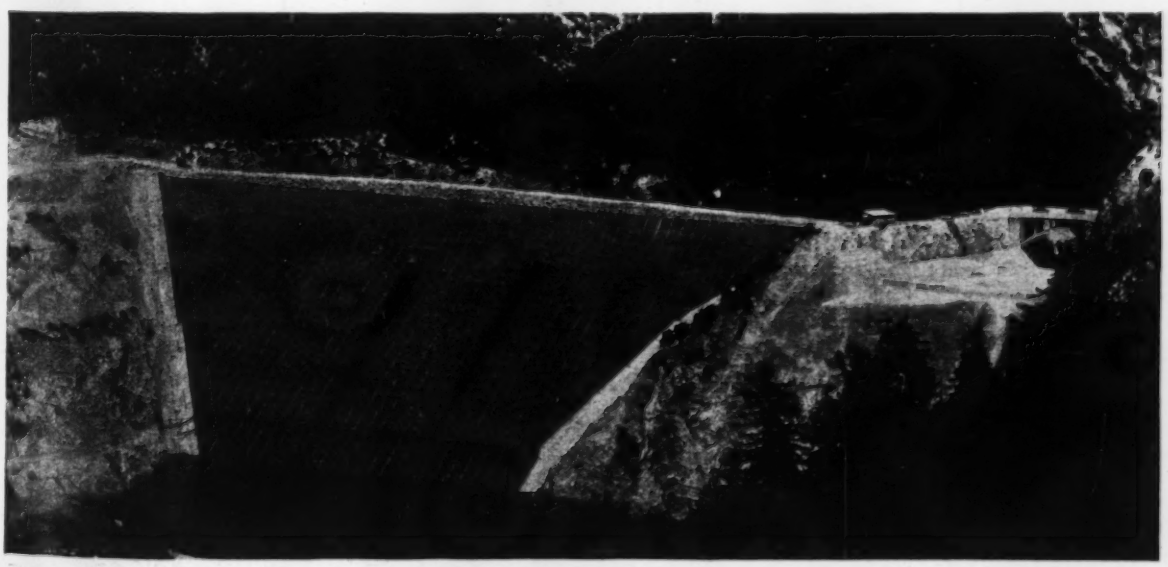
During the flood of March 1938, the dam was under nearly full water load with reservoir level 8.7 ft. below the crest of the dam. Just prior to this time, under a head of 206 ft., and a rising stage, the maximum leakage measured was 105 cfs. Later, under a head of 219 ft. and a falling stage, the leakage had decreased to 93 cfs. and still later in the year, when the head equalled that producing a leakage of 124 cfs. of the previous year, the leakage was only 11 cfs.

It is not possible to allocate the percentages of these decreases that should be credited to repairs, silting or the respective water-resistant qualities of the concrete and timber.

Settlement of the rockfill, which caused breaking up of the concrete slab, has been observed regularly. When the timber facing was completed, vertical settlements showed a definite trend toward approaching final settlement. Between the spring of 1935 and the spring of 1938 (subsequent to reservoir filling in the March 1938 flood) maximum settlement on the crest had amounted to 0.89 ft. Also, the yield normal to the facing resultant from water pressure, the so-called "dishing," amounted to 1.32 ft. This occurred in the lower third of the upstream face near the center.

At low reservoir stage recently the timber facing was examined, after three years of service, to observe the degree of deterioration. Some dry rot was found along contacts between timber and the concrete below; some was also found along the contacts between planks. In general, however, observations confirmed previous estimates that the life of the timber facing will be five to ten years with indications favoring the latter figure.

Average yearly cost of maintenance and repairs of the timber facing to date have been $0.0063 per square foot, or less than two per cent of the original cost.



A timber facing was put on San Gabriel Dam No. 2 in 1935 after settlement had cracked the concrete facing.

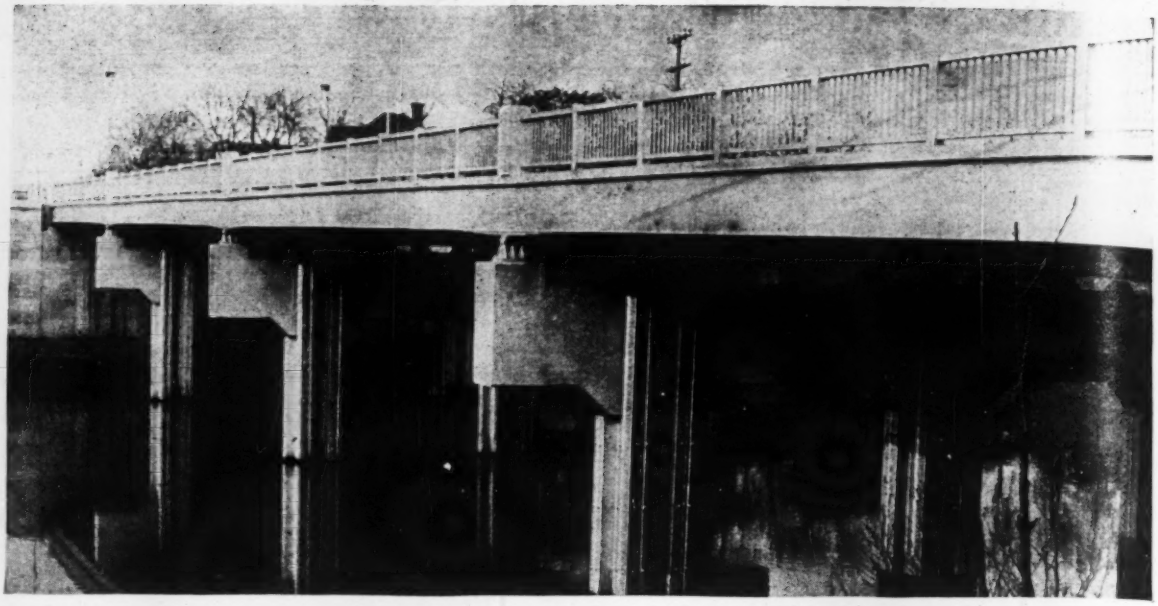


Fig. 1. Shafts of Z-piling driven for bearing into sand-gravel strata and filled with concrete support bridge.

Z-Piling for Bridge Pier Shafts

Louis K. Whitcomb, Jr.
Engineer Sales Promotion
Carnegie-Illinois Steel Corp., Pittsburgh, Pa.

***Contents in Brief**—For a sand-gravel foundation condition at a bridge site, Z-piling piers in the shape of a double cross are used, with piling driven deep for bearing and to prevent scour. The piers are concrete-filled above the sand-gravel strata to provide additional bearing and to give stability to the pile frame. Comparatively new in the highway field, Z-piling piers have been satisfactorily used on railroad and other construction.*

SELECTED after competitive bids with conventional gravity piers, a bridge over the Tuscarawas River at Dover, Ohio, has been built using Z-piling in the compact shape of a double cross as piers. The piling is driven for skin friction bearing in sand-gravel strata containing some clay, and the part of the Z-pile box above solid foundation material is filled with concrete for additional bearing and rigidity of the structure.

The new bridge, which replaces a three-span through truss structure with an 18-ft. roadway erected in 1893, has a 42-ft. roadway and two 8-ft. sidewalks carried on continuous rolled-section beams. The three river piers of three shafts each and one abutment consisting of three shafts joined by a thin reinforced concrete curtain wall utilize the Z-pile design. The other abutment, outside of the river bed, is a reinforced concrete cantilever wall on piles.

Pile length requirements for the job were determined by driving a test pile in its final position in one of the shafts to a resistance of 30 tons or to a depth of 34 ft. below the existing streambed, which is expected to provide a safe bearing value of 20 tons in the event of streambed scour of as much as 10 ft. A No. 7 double acting hammer was used driving to a resistance of 30 blows per inch for the last 3 in.

The total load per shaft at the river bottom is 248 tons, which can be safely carried by skin friction of the twelve piles. Furthermore, if this load is considered as being supported on the firm combined sand and gravel within the shaft area, the load per square foot would be only 10.7 tons, which is not too high for this type of material. Each pier shaft is made up of 8 pieces of copper bearing Z-sheetpiling ($\frac{1}{2}$ in. thick and weighing 57 lb. per ft. of pile and 38 lb. per lin. ft. of wall), and of 4 corners, fabricated to reverse the interlocks to make the closure. Piles are 63 to 65 ft. long except for the two at each side of each shaft, which are 8 ft. shorter to permit continuity of the reinforcing and concrete of the pier caps as shown in Fig. 2.

Old bridge used as falsework

The location of the new piers did not interfere with the concrete filled steel cylinder piers of the old structure, and by closing off all traffic on the bridge and approaches the

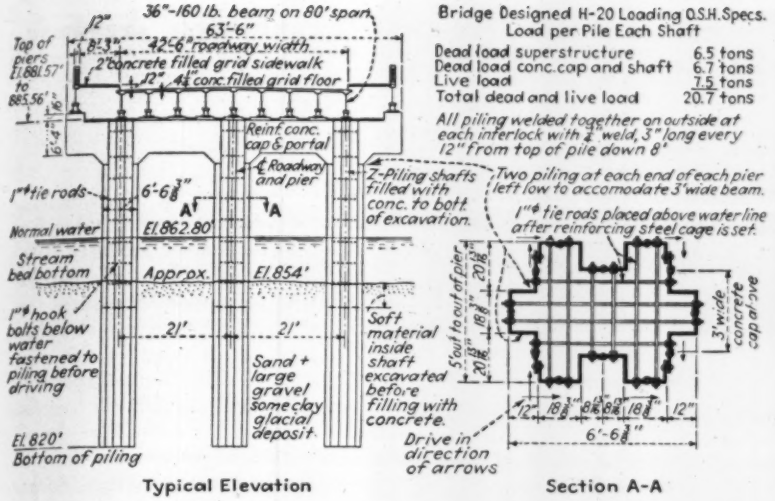

36"-160 lb. beam on 80' span
63'-6"
42'-6" roadway width
2" concrete filled grid sidewalk
12" 4½" conc. filled grid floor
Reinf. conc. cap & portal
℄ Roadway and pier
Top of piers El. 881.57' to 885.56'
1"ϕ tie rods
6'-6⅜"
A
A
Normal water
El. 862.80'
Stream bed bottom
Approx.
El. 854'
1"ϕ hook bolts below water fastened to piling before driving
21'
21'
Sand + large gravel some clay glacial deposit
Soft material inside shaft excavated before filling with concrete.
El. 820'
Bottom of piling
Typical Elevation
Bridge Designed H-20 Loading O.S.H. Specs.
Load per Pile Each Shaft
Dead load superstructure 6.5 tons
Dead load conc. cap and shaft 6.7 tons
Live load 7.5 tons
Total dead and live load 20.7 tons
All piling welded together on outside at each interlock with ¼" weld, 3" long every 12" from top of pile down 8'
Z-Piling shafts filled with conc. to bott. of excavation.
Two piling at each end of each pier left low to accomodate 3' wide beam.
1"ϕ tie rods placed above water line after reinforcing steel cage is set.
5' out to out of pier
20⅛"
18⅜"
20⅛"
3' wide concrete cap over
12"
18⅛"
6⅝"
6⅝"
18⅛"
12"
6'-6⅜"
Drive in direction of arrows
Section A-A

Fig. 2. Details of Z-pile piers. Concrete extends down only to solid foundation. Concrete cap beam, 3 ft. wide, passes through the shafts and ties them together.

contractor was given unobstructed use of the entire site. Investigation of the old trusses indicated that a 50-ton crane working on the old bridge was safe, and this method was used to avoid construction of falsework and shorten the construction period. Outside pier shafts were built by removing the sidewalk planking on the old bridge at the shaft locations. As the center shaft of each pier was practically on the centerline of the old structure a few stringers and the top chord diagonal bracing were removed to permit setting the new piling in place.

The only falsework required consisted of four wood piles at each shaft, driven into the river bottom in a square, braced on all sides and tied at the top to the old bridge floor to brace the entire frame. Inside the pile frame, and fastened to it, were two sets of templets, one just above water level and the other just below the final top elevation of the piling shaft. Each of these templet sets, made of oak and reused at each pier, was built with an inside and outside frame with a tolerance of ¼ in. over the theoretical cross-sectional dimensions of the steel sheetpiling shaft. The inside frame was fastened to the outside templet frame and cut free as the closing piece of piling was threaded into place. Accurate alignment was easily secured by this simple and inexpensive falsework and templet arrangement.

Prior to the driving of piling, 1-in. round hook bolts were fastened to the inside face of each piece, located so that after driving they would be between the water line and the bottom of the inside of the shaft. These hook bolts together with cross tierods above the water line securely anchored the concrete fill to the piling thus assuring that the concrete and steel piling would act as a unit.

Three days were required to place and drive the sheeting for each shaft, all piles being set up to form the complete shaft, then driven in the order indicated by the arrows on Section AA of Fig. 2. Each pile was driven about 5 ft., then the hammer moved to the next pile, continuing thus until all pieces were driven to the required depth.

Little excavation required

The soft river bottom material, overlying good sand and gravel foundation, was removed in the immediate area of the shaft before the piling was set, and in some instances a small additional amount was removed from inside the shaft after driving was completed.

After the steel sheetpiling was driven to grade, the sections were welded together at the interlock on the outside, using a bead 3 in. long every 12 in. for a distance of 8 ft. from the top of the piling.

Although the piling was fabricated from corrosion-resisting steel, the outside face of each piece was given a coat of acid-free bituminous paint, having a cement binder, to protect the steel from the industrial and mine waste water in the river. Gray paint to match the color of the concrete cap was applied to the outside of the shaft above the water line after completion of the concrete work.

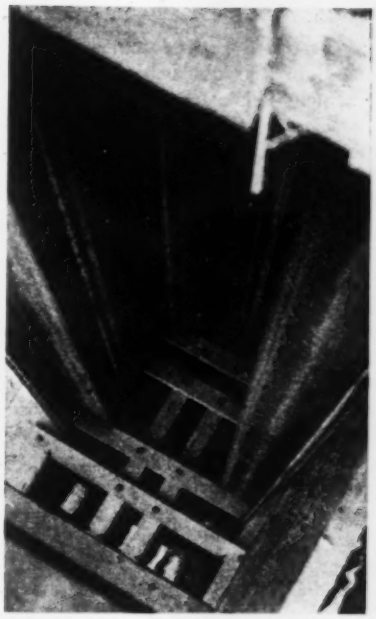


Fig. 3. Two templets, one near the water and the other near the top of the shaft, held the Z-piling plumb and in position while it was being driven.

When the piles had been driven to grade and the shaft cleaned of soft material, a prefabricated cage of reinforcing steel was lowered into the shaft extending from the river bottom to normal water level. Above this point 1-in. round tierods connect opposite piling sections at 4-ft. intervals. Forms for the concrete cap sections were fastened to the steel sheetpiling units through the tierod holes. Concrete work was carried on in two stages, first filling the steel shaft to the bottom of the beam haunch and then, after the reinforcing steel required to develop the beam and portal action had been set, placing the balance of the concrete.

The crane which handled the pier shaft construction was used for erecting the new bridge floorbeams. Since the new pier top width was 63 ft. 6 in. overall as compared to 35 ft. for the old bridge, there was sufficient room on each side of the old structure to place half of the 36-in. 160-lb. wide flange beam required for the two 80-ft. center spans and 70-ft. flanking spans at either end which make up the 300-ft. length of the bridge. Two of the new framing beams were then used for dismantling the old bridge structure, being lashed

together and jacked up against specially prepared seats fastened to the bottom of the old bridge sidewalk brackets. This permitted the old trusses to be taken down by panels without the use of falsework. After the old superstructure was completely removed, the pedestals for the new beams were placed and the beams positioned with diagonal channel cross bracing welded in place to the beam webs, holding the floorbeams at 5-ft.-3-in. spacing.

The 4½-in. grid pattern flooring was laid transversely over the floorbeams, welded, and filled flush with concrete. Two-inch steel grid sidewalk units were welded to the sidewalk stringer beams and filled with concrete after the erection of the floor units and railing. The bridge railing is steel construction, the main posts being two angles welded together and capped. Top and bottom rails are thin-wall tubing and the fillers are 1-in. square rods welded to the rails. The main posts are welded to the steel of the sidewalk flooring.

Several advantages in Z-pile use

The new type piers, such as are used in the Dover bridge, afford many advantages. They can be designed to any desired size or shape to meet engineering requirements and easily completed to any desired depth. The interlocking units of piling are much more easily driven than cylindrical shells. Each piling section can be driven to a predetermined loading or to a predetermined depth, thus guarding against scouring action and at the same time making possible a pier design in which each individual piece of steel in the pier carries its proportional share of all bridge and pier loads. Cofferdams are unnecessary and construction can proceed under adverse river stages, thus reducing construction time. Pier foundation loads are held to a minimum. Construction can all be accomplished from the water surface, either from floating equipment, trestles, or from the old bridge.

Design and construction was under the supervision of George Arnold, county engineer of Tuscarawas County, Ohio. Wendling Bros. Construction Co., Dover, Ohio, constructed the bridge, fabrication of the structural members being done by the Burger Iron Co., Akron, Ohio. PWA assisted in the financing and construction of the project.

Field-Spliced Precast Concrete Arches

Joseph J. Weiler
Architect and Engineer, Madison, Wis.

Contents in Brief—*In building a fireproof garage for road machinery, precast arch frames and roof slabs of light-weight concrete were adopted in preference to steel truss or concrete beam construction. To facilitate shipment and erection, each frame was made in three sections, spliced after erection. Tests of splices with simple vertical joint showed high strength.*

A FIREPROOF machinery-storage garage building 40 x 122 ft. that the Columbia County Highway Commission wanted to build at Cambria, Wis., was originally planned as a structure of steel columns, steel crescent trusses of 40-ft. span, wood roof and suspended plaster ceiling to protect the underside of the roof construction. This design was not entirely satisfactory. On the other hand, a precast concrete roof slab of light-weight aggregate on precast beams would have required a beam section so large that handling the units in the field and supporting them on the concrete-block walls would have been too difficult. A precast arch frame as shown by the accompanying drawings and views provided a logical solution.

Frame construction reduced the effective beam span to 24 ft. or less, and the negative moment taken by the legs of the frames produced deep sections where they least affected the movement of equipment within the building. The concrete legs also simplified the construction of the concrete block walls. Precast spandrel beams doweled and set in cement grout on brackets cast on the sides of the arch frames formed the head of the opening for the garage doors, which occupy much of the front.

The arches are braced together in the roof plane by diagonal tension rods. At the eaves they are tied together by the spandrel beams, which are anchored to each frame by pairs of rods embedded in grooves and holes in the top surfaces of the beams and encased in cement grout, which also forms a bed for the roof slabs.

In order to facilitate erection of the arch frames, each was made in three sections, with joints near the inflection points of the horizontal member. The length of this part was also affected by the problem of trucking the units 50 miles from Madison to Cambria. It was therefore necessary to field-splice the sections.

For ease in handling, 1-in. square rod eyebolts were cast in each arch section at its center of gravity. The same eyebolts furnished convenient

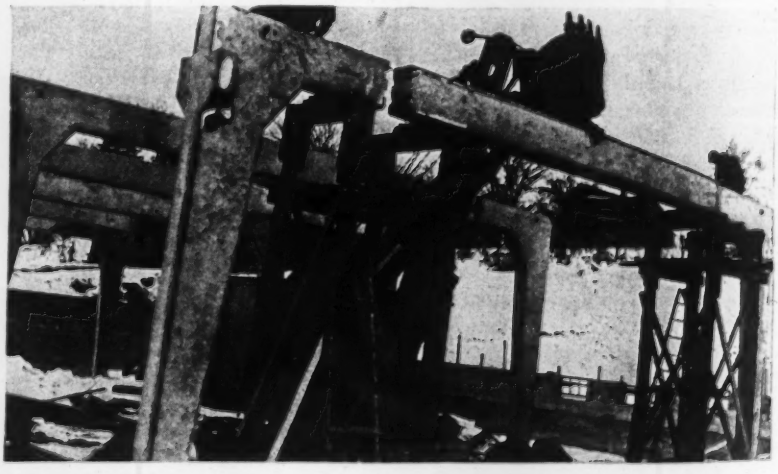


Fig. 1. The arch frames were made in three pieces for ease in truck transport and erection and were spliced in the field by welding the corner rods, inserting a stirrup and filling the joint with concrete.

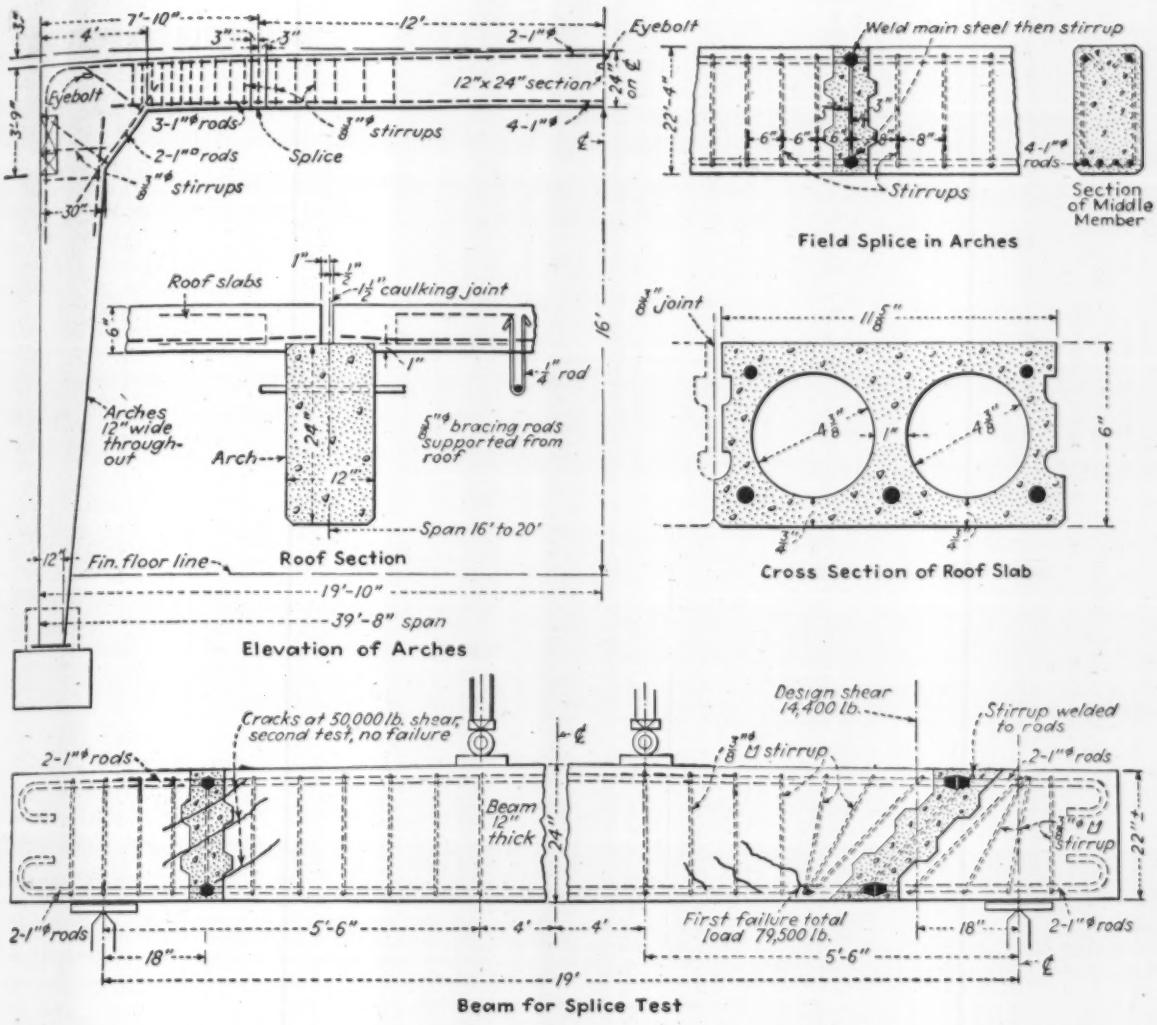


Fig. 2. Details of the precast frames and hollow roof slabs used in the garage construction and the splice test specimen.

points of attachment for the diagonal roof bracing.

In erection, each of the two legs of an arch frame was set up, with its projecting end supported by a timber support under the splice, and then the middle section was set on the same support, ready for splicing the arch. When the three units were lined up, the four outside corner rods of the arch reinforcement were butt-welded and a ⅜-in. U-stirrup was welded to the main steel on the center line of the splice. Then the joint was filled and tamped with concrete. A small canvas tent was placed around the joint to protect it from freezing, as the work was done during the cold months of the past winter; steam was supplied to this tent for several days to speed the curing of the joint.

After erection and completion of the arch frames, the concrete block walls were laid up, to the under side of the roof slabs, and along the front of the building the precast spandrel beams were set in place as already mentioned and tied to the arches. The hollow 6-in. roof slabs (Fig. 2) were bedded in mortar on the frames, and the spaces between adjacent slabs were grouted to produce an even roof surface. Roofing of jute fabric and asphalt was laid over the slabs.

Light-weight concrete used

All parts of the structure were made with Way-Lite, a light-weight aggregate of blast furnace slag. The resulting concrete weighed about 100 lb. per cu.ft. and had a cylinder compressive strength of about 3,000 lb. per sq.in. and a beam strength (modulus of rupture) of 250 lb. per sq.in.

The design for the garage was made by Law, Law & Potter, architects, of Madison. The writer acted as designer and engineer. The arches were fabricated by the Madison Silo Co. and erected by the maintenance forces of the Columbia County Highway Commission.

After the simple vertical splice had been decided upon, a load test was made on a beam including both a vertical and a diagonal splice as shown by sketch in Fig. 2 to provide additional information. The beam failed at the diagonal splice under a total load of 79,500 lb., slightly over 2½ times the design load; the vertical shear at the splice at failure was about 40,000 lb. The vertical splice was not injured in this test, and in the second test it sustained without failure a shear of 50,000 lb., the capacity of the machine, which was 3½ times the design shear of 14,400 lb.

DELAWARE AQUEDUCT VIII

Reservoirs and Dams

***Contents in Brief**—Three reservoirs and dams with connecting tunnels will be constructed as part of the Delaware water supply system. This article deals with the physical, hydrological and geological characteristics of the watersheds, and also gives the design of the Merriman Dam. This dam is a rolled earth structure with caisson cutoff walls. See footnote for a list of previously published articles in the Delaware series.*

THE DELAWARE WATER SUPPLY project includes three impounding reservoirs— (1) Rondout reservoir on Rondout Creek, a tributary of the Hudson; (2) Neversink reservoir on Neversink River, a tributary of the Delaware; and (3) East Delaware reservoir on the East Branch of the Delaware River. (See map *ENR*, Apr. 25, 1940, p. 587). From East Delaware reservoir and from Neversink reservoir the water will be conducted through tunnels into Rondout reservoir from which the Delaware aqueduct will convey the mixed waters to New York City.

Rondout and Neversink reservoirs with the 6-mile connecting tunnel will be constructed in the first stage of the Delaware development. The second stage of the project, comprising East Delaware reservoir and its 26-mile connecting tunnel will be put under construction when it becomes necessary to meet increasing demands for water.

The three watersheds lie on the westerly and southwesterly slopes of the Catskill Mountains, adjoining the watersheds of the Catskill system. This region is sparsely populated, largely forested, free from limestone, and characterized by high mean annual precipitation varying from 40 to more than 60 in.; steep slopes predominate and the runoff is rapid.

The region has been heavily glaciated and the preglacial rock valleys are deeply buried under glacial and waterborne deposits, some of which are waterbearing. The underlying rocks are sandstones and shales of the Catskill series of generally good quality except for superficial open joints and surface weathering. Valleys suitable for reservoir sites are long and deep without low saddles in the rim; each requires but a single dam.

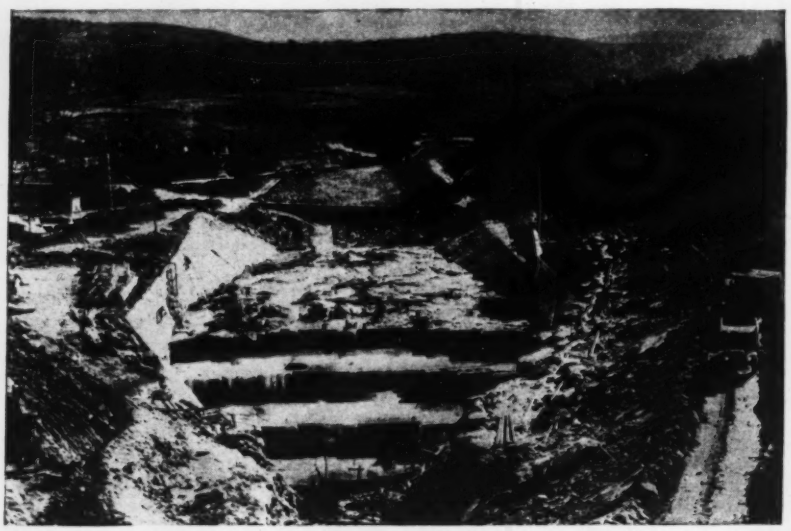


Fig. 1. Stilling pool below the outlet of the diversion and spillway tunnel. The serrated steps are 10 ft. high. Top step is 10 ft. above top of tunnel.

Conditions at damsites

Conditions at the three damsites are generally similar. The valleys are deep, steep-sided and flat-bottomed, with the bedrock overlain by 100 to 180 ft. of irregularly deposited sand, gravel, boulders and clay in various combinations, with considerable water in certain horizons. In each case the indicated design is an earth dam with concrete cutoff to bedrock, a valley-side spillway founded on rock, and a diversion tunnel in bedrock around one end of the site to handle streamflow during construction. In each case suitable material for the construction of an earth dam is available near the site. High rates of flood runoff call for large capacities both for the diversion tunnels during construction and for the spillways and waste channels of the finished reservoirs.

At Rondout reservoir it is necessary to provide storage for water supply only. The Neversink and East Delaware reservoirs, however, are on tributaries of the Delaware River, and to satisfy the terms of the U. S. Supreme Court's decree confirming New York City's right to divert water from this interstate stream, these reservoirs must also provide sufficient storage to permit release of compensation water into the stream during dry periods; the maximum rate is fixed at 0.66 cfs per square mile of watershed tributary to these reservoirs.

When both Neversink and East Delaware have been developed, this will require releases up to a maximum rate exceeding 300 cfs or nearly 200 mgd. To supply this compensation water a very substantial amount of storage in Neversink and East Del-

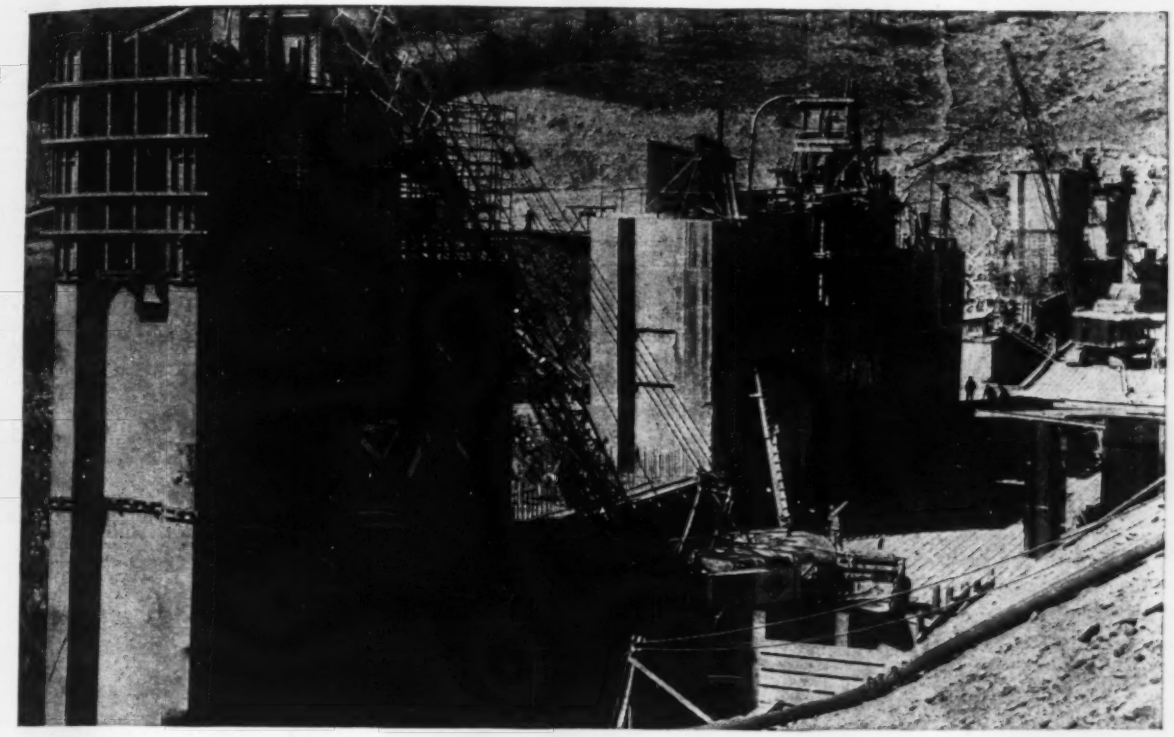


Fig. 2. Cutoff wall for Merriman Dam is a series of concrete caissons, sunk to rock.

aware reservoirs will be required in addition to that necessary for water supply.

The controlling dimensions of these reservoirs and their dams are shown in the accompanying table.

Merriman Dam details

The first of the dams to be built, that on Rondout creek, at Lackawack, N. Y., forming the Rondout reservoir, is now under construction. It has been named as a memorial to the late Thaddeus Merriman whose vision, courage, outstanding ability and indefatigable energy not only conceived the Delaware project but overcame all obstacles and made its execution possible.

Rain gages have been maintained in the Rondout drainage area above Merriman Dam for 33 years. During this period the annual precipitation based on the unweighted average of these gages has varied from a minimum of 39 in. to a maximum of 61.6 in., with an average of about 49 in.

Streamflow records for stations a short distance downstream of the site are available for the period 1907 to date. Over this period the average flow has been 2.3 cfs per square mile. The maximum recorded flow occurred on Aug. 26, 1928, when a peak of approximately 270 cfs per square mile was reached.

Geological conditions

Bedrock at the site lies under a deep mantle of overburden about 20 to 30 ft. thick on the right side of the valley, about 40 to 60 ft. thick on the left side, and about 180 ft. thick for a considerable width near the middle of the valley floor. On the sides of the valley the overburden is mainly glacial till, but under the valley floor the overburden consists of a complex and somewhat erratic assembly of glacial, waterborne and lake-deposited materials, sands, gravels, boulders, silt, clay and glacial till.

The bedrock is a medium to coarse grained sandstone (graywacke sandstone belonging to the Ashokan formation of the Catskill series and commonly known as bluestone used for sidewalk flagging, curbs, etc.). This rock is strongly and nearly level-bedded, breaking easily along the bedding, and associated with shaly layers. The upper 5 to 17 ft. is considerably decayed along the bedding planes, but the underlying rock is sound. Water-bearing joints in this sandstone supplied limited artesian flow to the surface 180 ft. above.

The evidence developed by the borings and the two exploratory caissons (to be described in a subsequent article) was that there are several distinct and poorly connected waterbearing horizons, notably that in the top few feet of the rock. The overburden is dominated by till masses and sands and gravels not readily deformed, and the finer sediments are compact and deeply buried. The whole mass is stable and competent to support the dam. The evidence further indicated that below the top few feet of weathered and waterbearing rock, at a depth not exceeding about 200 ft. the bedrock is sound and essentially undisturbed but with some waterbearing joints and bedding planes. A tight cutoff to sound bedrock, and grouting of the bedrock was indicated. In general the indications of the preliminary exploration have been confirmed in sinking the remaining caissons.

The deep cover of glacial material on the left side of the valley affords material suitable for a rolled embankment. The material upstream from the dam, mainly glacial till with some deposits of clay, sand and gravel, has been selected as most suitable.

The Merriman Dam (Fig. 3) will

be a rolled earth embankment with a concrete cutoff wall to rock. It will be about 200 ft. high above streambed, with a crest length of 2,500 ft. The top width is 60 ft., and the bottom width at the maximum section is about 1,400 ft. exclusive of the loading embankments on the toes.

Upstream slopes vary from 1 on 2½ to 1 on 4, downstream slopes are 1 on 3 and 1 on 4. Both slopes will be covered with heavy blankets of selected rock, and a heavy rockfill will form the downstream drainage toe. Rockfill in the creek bed below the dam will drain the downstream blanket and the drainage toe. The stream-control cofferdam, of identical construction with the main dam, forms the upstream toe. Both toes will be loaded with embankments of earth and rock spoil compacted only by the grading and hauling equipment. The upstream rock blanket will be finished with heavy hand-set stone paving, the downstream rock blanket covered with soil over 2 ft. of earth, and grassed.

It is intended that the rolled embankment shall be impervious throughout. The material will be so graded that there will be a gradual and as nearly as possible a uniform increase in the proportion of coarse material from the central portion which will contain the largest proportion of fines, to the outer surfaces which will contain the largest proportion of sand and gravel. Materials of different textures will be mixed in the borrowpit as necessary, screened to 5-in. maximum size, moistened or dried as required for most effective compaction, spread in 7-in. layers and compacted with sheeps-foot rollers.

It is intended to compact the material to the maximum dry density practically attainable and selection and grading of material, moisture content and density will be subject to continuous laboratory control. It is anticipated that about 12 trips of a single roller drum will be needed. Payment will be adjusted to the number of trips actually required. The foundation area will be stripped of soil and, immediately before placing embankment material, plowed, harrowed, and moistened if necessary.

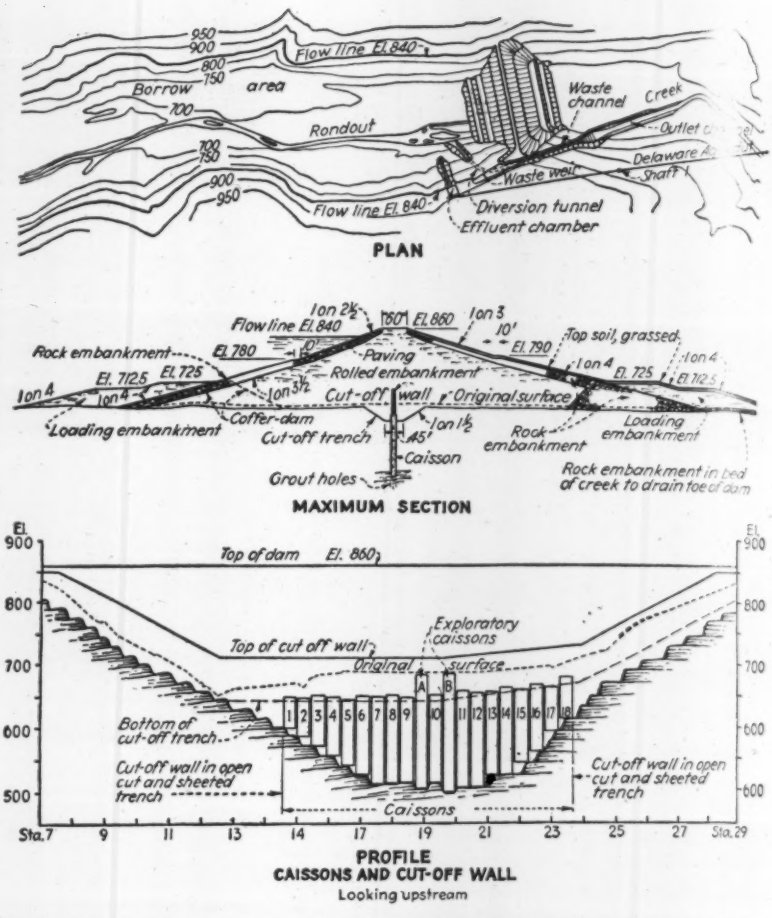


Fig. 3. Merriman Dam is a rolled earth structure with a caisson cutoff wall. The earth embankment is 200 ft. high with rockfill blankets on both slopes. The concrete caisson wall goes down to rock through 180 ft. of overburden.

Concrete cutoff wall

A concrete cutoff wall will be carried into sound rock for the entire length of the dam, and the underlying rock will be grouted. In the shallower overburden this wall will be built in open cut; in somewhat deeper overburden, a sheeted trench will be used below the bottom of the cutoff trench. In the deepest overburden for a length of about 900 ft., the wall will be constructed by sinking a line of 20 rectangular reinforced concrete caissons from the bottom of the cutoff trench to sound ledge-rock and sealing the caissons to the rock and to each other.

The cutoff trench in each case will have a 45-ft. bottom width and it will be from 30 to 40 ft. deep. The cutoff wall will be carried to a variable height above the original surface, but not as a corewall to the top of the dam (Fig. 3). The trench will be refilled with material selected and compacted as in the core of the dam, except that next to the wall the earth will be spread in 3-in. layers and compacted with pneumatic or other mechanical tampers. Use of 3-in. layers and mechanical tamping will also be required along the junction between the embankment and the surface profile and at other places.

The dam, including the refill in the cutoff trench and the loading at the toes will involve about 6,900,000 cu.yd. of rolled and other earth embankment and 1,000,000 cu.yd. of rock embankment.

Material for the rolled embankment will be obtained for the most part from the left side of the valley where the cover of glacial material is deepest. Earth borrow will be taken only within the reservoir flowline, but to avoid inviting leakage through the joints of the weathered surface rock, never to such depth as to expose bedrock. Quarries will be above the flowline for the same reason.

The caisson-cutoff wall is well advanced toward completion. The cais-

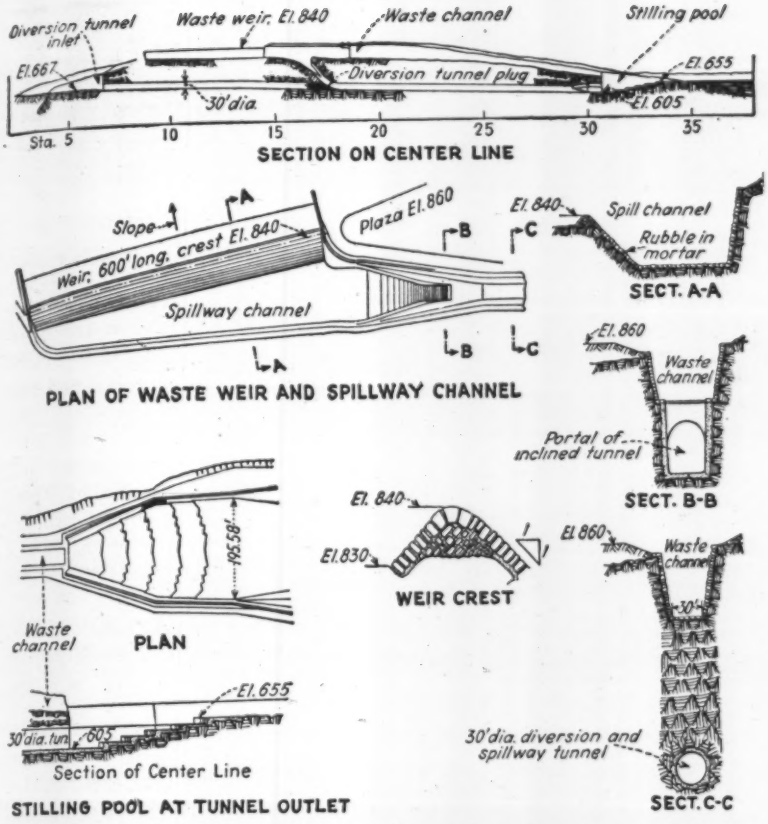


Fig. 4. Waste weir, spillway channel and diversion tunnel details. The 30-ft. tunnel will divert Rondout Creek around the site during construction of the dam, and carry floodwaters spilling over the weir after completion.

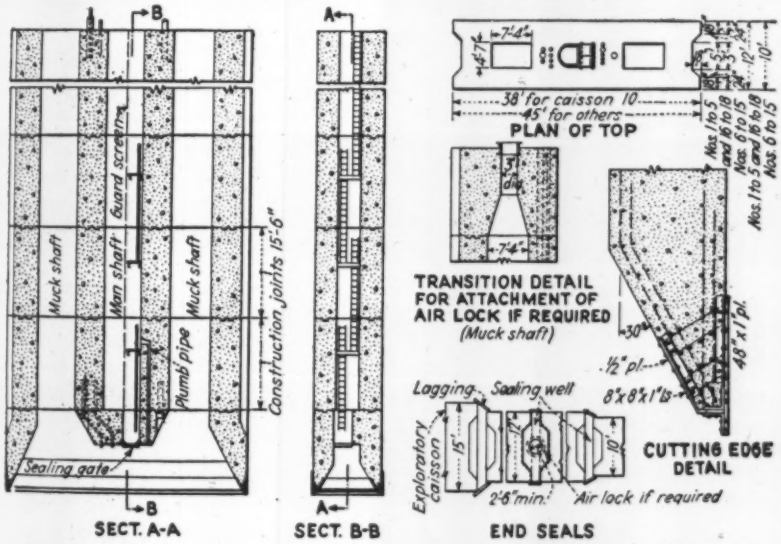


Fig. 5. Contractor's design for caisson construction. About 18 such caissons will be sunk to complete the deep concrete cutoff wall for Merriman Dam.

sons, except the two exploratory caissons and one other, are 45 by 12 ft. in plan in the deeper part, and 45 by 10 ft. at the shallower ends of the cutoff. The deepest caisson penetrated 136.5 ft. of overburden below the bottom of the cutoff trench, and is 169 ft. in overall height; the shallowest caisson penetrated 53 ft. and is 76 ft. in height.

End clearances between caissons is nominally 18 in., and an 18-in. vertical recess in each end of each caisson, provide sealing shafts which are being excavated and filled with concrete after the caissons have been sealed to rock and the groundwater controlled. In every case excavation is being carried below the cutting edge to sound rock. Such excavation through overburden and open-jointed and water-bearing rock has been carried to a maximum depth of 26 ft. below the cutting edge and 150 ft. below the bottom of the cutoff trench. These excavations are being completely filled with concrete to above the cutting edges and eventually the working chambers will be filled with concrete, the underlying rock grouted, and the man and muck shafts filled with selected and compacted earth. At the end of October twelve of the 18 caissons other than the exploratory caissons had been completed and sealed to sound rock.

Two exploratory caissons in the line of the cutoff were sunk, in 1937–38, under an earlier contract, at the deepest part of the bedrock valley where the original groundwater pressure was about 80 lb. per sq.in. Pumping from these caissons, from a deep well, and from the subsequent caissons as they have been sunk has so lowered the ground water that 93 per cent of the 1,882 lin.ft. of excavation below the bottom of the cutoff trench, completed to Oct. 30, had been done under free air, and maximum air pressure required at any time was 38 lb. per sq.in.

The caissons other than the exploratory caissons are shown in Fig. 4. To gain additional weight for overcoming skin friction the contractor has reduced the size of the muck shafts in many of the caissons to 3-ft. diameter throughout. As required by the specifications the men are hoisted in and out of caissons deeper than 60 ft. and the manlocks are lighted, heated when needed, and are large enough to provide seats for 20 per cent more men than one full shift.

Diversion tunnel and outlet

For the protection of the site during construction of the dam a cofferdam of about 50-ft. maximum height and placed to form the upstream toe of the main dam has been built. Rondout Creek is diverted through a tunnel, 2,450 ft. long, concrete-lined to a 30-ft. diameter, which was driven through the rock of the right abutment (looking downstream). At the outlet the normal 30-ft. circular sec-

CONTROLLING DIMENSIONS OF DAMS AND RESERVOIRS OF THE DELAWARE SYSTEM

Reservoirs (in order of construction)	Elev. ft. above Mean Sea Level: Spillway crest	Elev. ft. above Mean Sea Level: Top of dam	Max. height of dam above streambed ft.	Approx. crest length of dam ft.	Approx. max. depth of overburden ft.	Approx. length of reservoir miles	Available capacity mg.	Drainage area sq. mi.	Storage mg. per sq. mi.†	Estimated yield for safe draft mgd.
Rondout	840	860	200	2,500	180	7.2	49,000	95	516	90
Neversink*	1,440	1,460	200	2,700	150	5.3	36,000	93	387	110
E. Delaware*	1,280	1,300	200	2,700	100+	18.8	147,000	372	395	410
							232,000	560	414	540

* All data for Neversink and East Delaware reservoirs are tentative.
† Since both Neversink and East Delaware reservoirs will discharge into Rondout reservoir, storage per square mile can be equalized between these reservoirs to any extent desired.

tion of this tunnel is gradually expanded, so that in the last 75 ft. it is double the initial area. This has been done to reduce the velocity of discharge.

The discharge, which is submerged, is into a stilling pool of a novel design (Fig. 1), worked out by means of model experiments. Immediately upstream of the tunnel inlet, stop-log grooves have been provided so that if the city is short of water prior to the completion of the dam, stop-logs can be placed to raise the water above the sills of the outlet gates and deliver it into Delaware Aqueduct.

The maximum flood flow in the 28-yr. period of record was nearly 27,000 cfs, the second highest about 18,000 cfs. The estimated capacity of the diversion tunnel with reservoir at El. 715 (5 ft. below the top of the cofferdam) is 20,000 cfs. Reservoir storage below this elevation will be about 5,700 acre-feet. A depressed section of the cofferdam located upstream from the toe of the main dam and across the channel of the creek is of rock embankment and steel sheet-pile construction with its top at El. 715; this will serve as an emergency spillway in the event of an exceptional flood during the early stages of construction of the main dam. The contract for the dam provides for a separate bid price to cover the costs of stream control other than the cofferdam and diversion works built under a prior contract.

Spillway tunnel and channel

The spillway will be of the side channel type with a 600-ft. weir (Fig. 4). With a depth on the crest of about 12 ft. leaving a freeboard to the top of the dam of about 8 ft. it will pass 100,000 cfs. The overfall will be into a flat-bottomed spillway channel excavated in rock and lined and paved with rubble masonry.

From the downstream end of this channel a transition section will lead into a tunnel inclined on a slope of 45 deg. This will be concrete-lined to a 30-ft. diameter, and connected by an easy vertical curve with the 30-ft. diversion tunnel. On completion of the dam, gate house, etc., the diversion tunnel will be closed with a massive concrete plug at its intersection with the inclined tunnel. The diversion tunnel below the plug will then become a part of the spillway tunnel.

The spillway tunnel will have a capacity of 50,000 cfs with a velocity in the 30-ft. section of 70 fps. As further provision for extraordinary floods an open waste channel will be excavated in rock directly over the spillway tunnel. This will extend from the portal of the inclined tunnel to the stilling pool previously described as at the outlet of the diversion tunnel. The waste channel will have a further capacity of about 50,000 cfs, making a total spillway and wasteway capacity of about 100,000 cfs or about 1,000 cfs per square mile.

Exploratory caisson data and control of groundwater at the Merriman Dam will be described in an article to follow.

Other Articles in the Series

A Billion Gallons Not Enough.....Apr. 11, p. 513
Routing the Tunnel.....Apr. 25, p. 584
Designing the Aqueduct.....May 9, p. 656
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Bridge Steel Corrosion

MEANS FOR MEETING the destructive effect of corrosive substances that come into contact with bridges and other steel structures on railways were reviewed in a report presented at the recent meeting of the American Railway Bridge and Building Association in Chicago.

A comparatively novel treatment is the metallizing of steel structures, or the coating of the members with non-corrosive metal instead of paint. This was noted as having been used on at least three railroads. After sandblasting of the surface, the coating is sprayed on in an atomized form by compressed air. Several railway bridges have been given a zinc coating in this way, one road reporting a cost of 50 to 65c. per sq.ft. A smokestack was given an inside coating of lead and an outside coating of aluminum. While these coatings have been successful, so far, they are too expensive for general use, unless their life should prove to be proportionately longer than that of paint.

Some railroads are experimenting with non-drying paints that retain their "oily wetness" as long as the paint lasts. Drying paints were stated to be never entirely moisture-proof, the best being about 98 per cent at the start. This type gives its best service when applied to clean steel. On the other hand, uncleaned and corroded surfaces appear to be most effectively protected by non-drying paint films, which, with their condition of "oily wetness," penetrate rust completely.

Moisture pockets often formed by structural details present a difficulty with paints of the drying type, but with non-drying paints they serve as reservoirs for liberal amounts of the coating. The report states that some railways find reasonable success in the use of the drying type of paints on steelwork subject to attack by brine and gas, while others have but poor results. The use of non-drying coatings appears to be gaining favor for parts affected by brine.

Oxy-Acetylene Equipment Expedites a Water Main Repair

IN A RECENT EMERGENCY JOB in Hammond, Ind., a diversion line for an important water main was constructed across the Calumet River, using oxy-acetylene welding, cutting, and even heating, to complete the job overnight. Speed was of major importance in installing the emergency line, in cutting away the elbows in the main line where it descended into the river banks, and in tying in the main line to the diversion line.

The emergency arose with the discovery that a 36-in. cast iron water main had cracked and was admitting river water and sewage where it passed beneath the Calumet River. The 35-year-old pipe was embedded 25 ft. below the river bottom at this point. In a first attempt to overcome pollution inflow, the water pressure in the pipe was increased in hope that seepage would be thus overcome. The result, however, was that the river boiled with water expelled from the pipe. City engineers then decided to replace the broken section with new pipe, which made necessary the installation of the temporary diversion line across the Calumet River bridge.

Diversion pipe laid first

In order to have everything in readiness before shutting off the water, the diversion line of 24-in. steel pipe was laid across the bridge to within a short distance of the main line where the tie-in was to be made at both ends by means of 36 to 24-in. steel reducers. These reducers were fabricated on the job by welding.

Fig. 1. Elbow ready for removal in order to join the main line with the diversion pipe.

A 20-ft. length of 24-in. pipe was then welded to each of the reducers.

At night when water service demands were at a minimum, a hole was cut in the two 36-in. elbows in the main line, one on each side of the river bank, using a cutting blowpipe and also a steel welding rod as a starting rod to assist the cutting action. Through the hole a pump was inserted to remove all water that remained in the elbows. The operator then severed the elbow by a circumferential cut. By means of the heavy-duty welding blowpipe, the lead calking in the bell-and-spigot joint of the elbow was melted out so that the elbow, thus freed, could be removed by block and tackle.

Main pipe connection

The main pipeline was then open and ready to be joined to the reducers and the diversion line. After measuring the distance between the lines, the 20-ft. length of pipe that previously had been welded to the reducer as an extension was cut to the correct length, and the reducer section was fitted into place and the tie-in made by couplings. The entire job was completed and water was flowing through the diversion line early the next morning.

The information and photographs used in this article were furnished by Linde Air Products Co., New York.

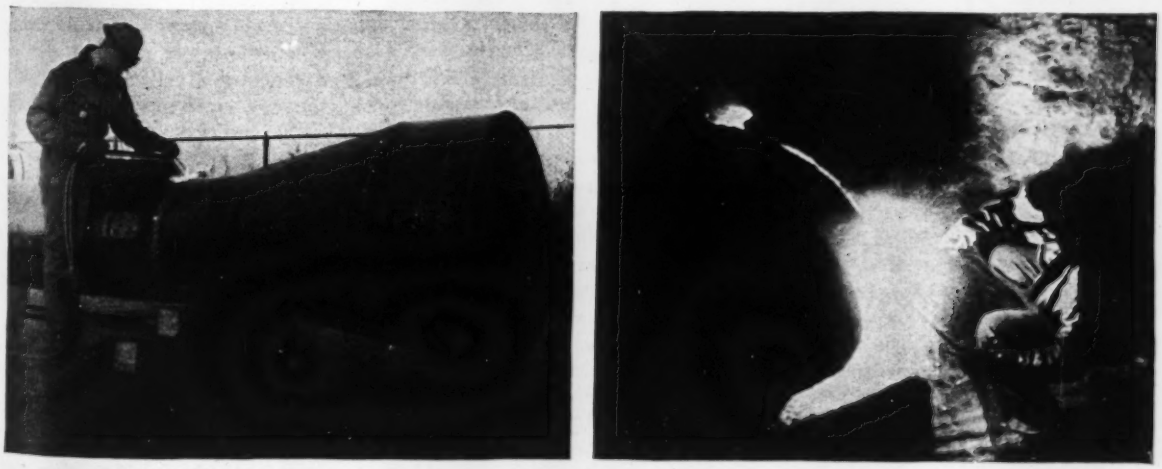


Fig. 2. Welding a 36 to 24-in. reducer to the 24-in. pipe (left); at the right is shown how the 36-in. cast iron elbow was severed with a cutting blowpipe and starting rod.

C. S. Johnson Co. photo

Fig. 1. Hillside batch plant for 3 aggregates and cement. Simple lever sets scales for either 27E or 34E paver batch.

Batch Plant for Mile-a-Day Paving

***Contents in Brief**—A 150-yd. 4-compartment (3 aggregate and 1 cement) bin used on one of the Pennsylvania Turnpike contracts batched all material required by a 34-E double drum and a 27-E conventional paver. A simple lever changed the scale beams to suit batch requirements for trucks operating to either paver. Best day's run required about 2,900 tons of aggregate and 2,760 bbl. of cement.*

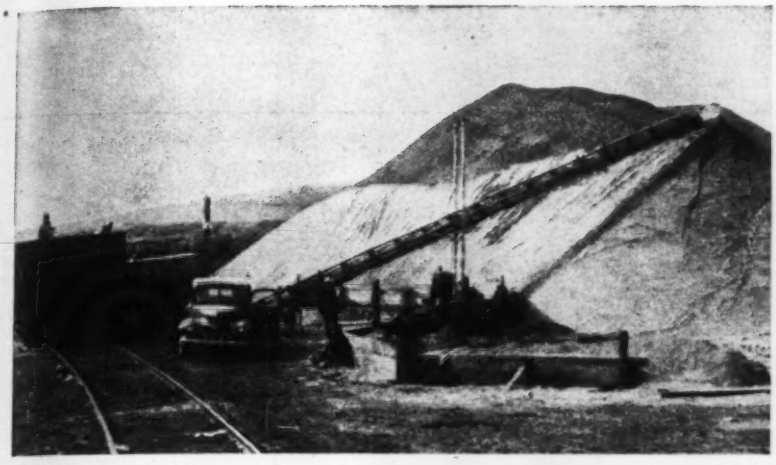
PAVING 6.84 miles of four 12-ft. lanes of 9-in. uniform thickness pavement on the Pennsylvania Turnpike's rush schedule required a lot of material, rapidly batched. On the Adam Eidemiller contract, near the westerly end of the road, a single 150-yd. semi-portable plant batched the sand, two sizes of stone, and the cement required for different size batches for a 34-E and a 27-E paver. The two pavers were operated as entirely separate units generally working in opposite directions from the plant. Best day for the two pavers was 5,302 lin. ft. in two 7-hr. shifts.

The plant was located on the Turnpike at the center of the contract and near an overpass which gave direct access to a rail siding 1.2 miles away. The bin was set against one side of a borrowpit cut as shown in Fig. 1. A ramp, one end resting on the bin, permitted dumping all material from the top.

Cement came to the rail connection in hopper-bottom cars, and there was transferred through a bucket elevator and 50-bbl. cement silo to covered steel dump body trucks for delivery to the batch plant. Sand also was delivered by rail in hopper-bottom cars and transferred to a stockpile by a radial belt conveyor car unloader and stacker; any time after a required 12 hr. in the stockpile it was reclaimed and loaded by a bucket type, crawler mounted, truck loader for haul to the plant. Most of the crushed stone was produced in the contractor's own quarry, trucked 8 miles to the plant and dumped either directly into the bin or stocked nearby for later reloading by crane.

The batcher plant is a semi-portable type requiring partial dismantling of the hopper for transfer by rail. It has capacity for about 35 cu. yd. of sand, 30 cu. yd. each of two sizes of coarse aggregate, and 300 bbl. of cement. The four single material batchers, three aggregate and one cement, were each hung under their respective bin compartments and were arranged to discharge into a collecting hopper that brought the three aggregates together and discharged them into the batch box of the truck through a long, narrow discharge opening. The cement was confined in a separate chute within the larger collecting hopper, and the discharge opening of this chute was spaced to deliver into a cement batch box mounted on the body of the truck. The trucks were spotted under the composite collecting hopper so that the openings of the cement batch box was under the discharge opening of the cement chute, and all batchers were then discharged simultaneously.

Two operators handled the batchers, one at the sand and cement batchers and one at the two stone batchers. As a truck, equipped for two-batch hauling, pulled ahead after receiving its first batch, the two operators weigh



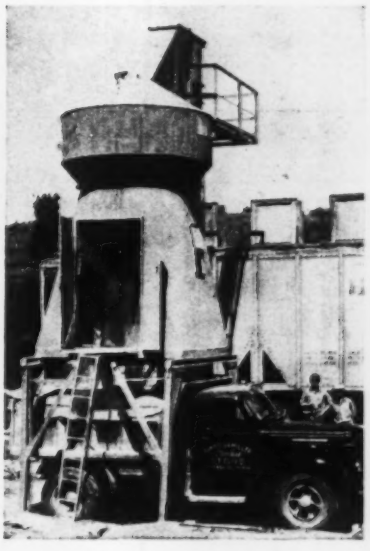


Fig. 2. Radial stacker unloads bottom dump cars into sand stockpile. Right: Cement is unloaded from car by screw conveyor and bucket elevator into 50-bbl. bin for transfer to bulk trucks.

all four materials simultaneously, each operator watching two scales at once, manually closing the gates at the proper weight, and immediately discharging again to give the truck the second batch. In this way, the operation of spotting the trucks overlocked with the batching operation so that the total time for the complete batching cycle was the controlling factor. Since the batchers could be filled and discharged in a little less than thirty seconds, the theoretical capacity of this plant was something in excess of 120 batches per hour. A test run indicated that more than 100 batches of irregular order 27-E and 34-E size could easily be produced.

Each batcher was equipped with two weigh beams, one of which was set for a 27-E batch and the other set for a 34-E batch. The trucks hauling to each paver were distinctively marked so that the operators could tell at a glance the size batch required for the truck coming into the batch plant and, by dropping the proper weigh beam into the scale lever system, they were able to change from one size batch to another with no appreciable loss in time.

Trucking, in and out

All material for paving came to the plant in medium size dump trucks. Rapid handling here was important as the trucks delivered a load of stone every 2 min., sand every 4 min., and cement every 10 min.

An I-beam ramp, 8 beams in the 30-ft. width and floored with 3-in. plank, accommodated three trucks at one time. Over the bin old rails resting on the I-beams formed a grillage through which the materials were dumped. The house over the cement bin is typical of hillside plants on the Turnpike, this protection reducing loss of cement when dumping and protecting it from moisture which might cause caking.

All other plants on the Turnpike required that trucks back under or drive through two or more bins to get the three aggregates and cement required for a batch. Here, however, the wide, drive-through roadway with ample access room permitted a truck to receive its entire charge for one batch at one stop, while a short move ahead permitted loading the second batch. This rapid loading resulted in substantial reduction in the number of batch trucks required, and led to real economies, especially on short hauls, where spotting and waiting time in multiple-stop operations amounted to a considerable portion of the total truck time.

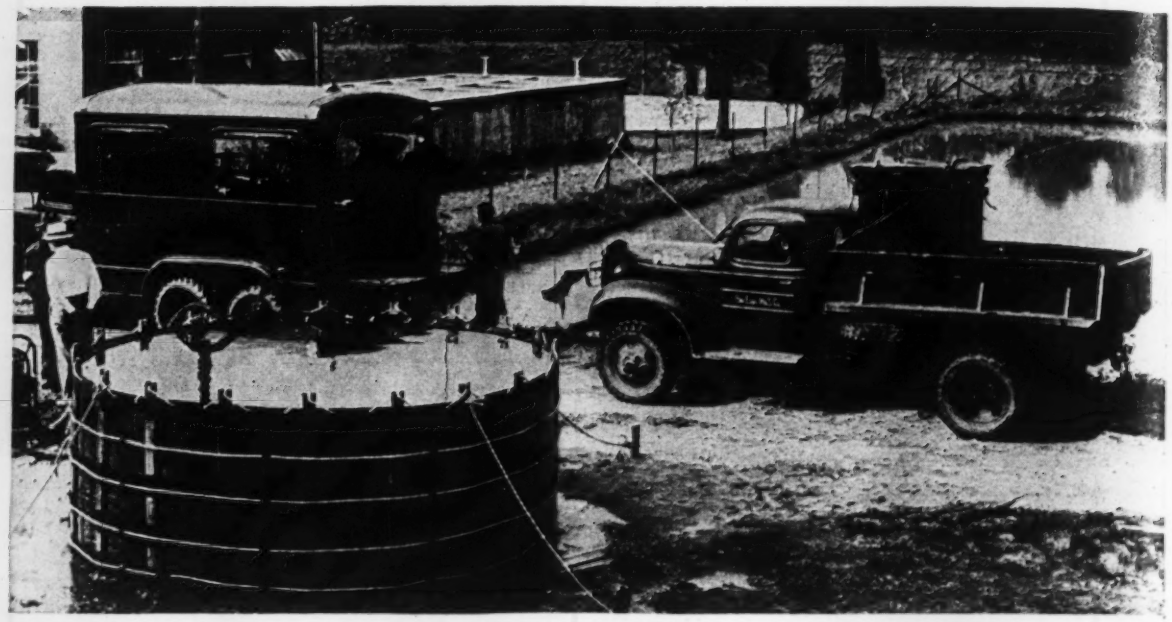
Personnel

For the contractor, Adam Eidemiller was in general charge, S. H. Showell was engineer and John Giffen foreman at the batcher plant. The Turnpike Commission was represented by Geo. T. Chapman, construction engineer, and Robert B. Yule, resident engineer for this section.

Weld Ductility Improved by Heating

Heating at relatively low temperatures for 1 hr. to 24 hr. or more produces a marked increase in the ductility of oxyacetylene welds, according to tests by J. R. Dawson and A. R. Lytle of Union Carbide & Carbon Research Laboratories, reported to the American Welding Society. The usual stress-relieving anneal, carried on at temperatures above 650 deg.C. is not superior and sometimes is inferior in its effect on ductility to prolonged heating at temperatures of 110 to 300 deg.C. (230 to 572 deg. F.). Heating to these low temperatures raised the ductility of welds of plain low-carbon steel by 30 to 50 per cent.

Test pieces of $\frac{3}{4}$-in. flange-steel plate welded with low carbon rod (single-vee butt weld made in three to five layers) showed initial values of about 34,000 yield point, 65,000 ultimate, 20 per cent elongation and 30 per cent reduction of area. Heating for 1 hr. to 300 deg.C. or 24 hr. or more to 110 deg.C. raised the elongation to 35 per cent and the reduction of area to 45 to 55 per cent, without affecting yield point or ultimate strength. Similar results were found in welds made with rods containing 0.35 per cent chromium, though in this case the 110-deg. treatment was less effective. Various alloy steels (except vanadium) showed like improvement. The effect of the improved ductility was also revealed by the fracture, which in the original condition was irregular or crystalline while the fractures after heat treatment were silky.



Army Engineer Corps photo

Fig. 1. Mobile purification equipment for the army. The collapsible reservoir is of canvas braced with wood.

Portable Purification Plant

Lt. Col. D. W. Gillette
Corps of Engineers, District Engineer, Wilmington, N. C.

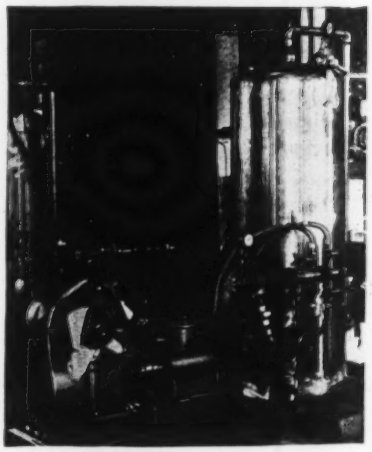
***Contents in Brief**—A water pumping and purification plant assembled in a compact, portable unit has been developed by the Army Engineers for field operation. Recently one of these units was rushed to a flood-stricken area where it demonstrated its versatility in supplying pure water to a town when the municipal supply failed.*

THE PORTABLE water purification unit is a striking example of the progress that is being made by the modern American army in the field of military and sanitary engineering.

In addition to being indispensable to the army, this equipment is of inestimable value to civilian populations in times of emergency as illustrated recently at Weldon, N. C. Floodwaters of the rampaging Roanoke River last August inundated the water plant in this town of 6,500 people, thus creating a serious situation. Within a few hours after this emergency was brought to the attention of the North Carolina Army Engineer District one of these units was on the scene furnishing 110,000 gpd of pure water.

The unit is divided into two sections. One section includes a gasoline engine driven pump, a hypochlorinator, chemical feed pots for the concurrent application of two water treatment chemicals such as soda ash and alum, a hypochlorite solution reservoir, and all necessary interconnecting piping, hose and accessories. The other section consists of a complete filter unit with all necessary piping, valves, gauges, etc., required for filtering and backwashing. The units are intended for field use and are appropriately constructed of rugged materials and handles for carrying purposes, all projecting handles being of such design that they may be folded in or telescoped when not in use.

The pumping unit consists of a self-priming, open impeller, centrifugal pump, with the pump impeller mounted directly on the extended engine crankshaft. Power is supplied by an air-cooled, 4-cycle, gasoline engine. The pump is capable of de-



Charlotte, N. C. Observer photo

Fig. 2. Portable purification units include a gasoline-driven pump, chemical feeding equipment and a filter.

livering continuously at least 55 gpm, against a total dynamic head of 50 ft. including 15 ft. of suction lift.

The hypochlorinator consists of a belt-driven gear unit and diaphragm assembly capable of pumping from 10 to 50 gal. of hypochloric solution per 24 hr. The diaphragm is of soft reinforced rubber and the pumping chamber hard rubber. The chlorinator is operated to provide a residual chlorine content of 1.0 ppm in the water after 10-minute storage in an open vessel.

The filter unit is provided with pure quartz sand and gravel, the filtering medium consisting of a 4-in. layer of sand on top of an 18-in. bed of gravel.

The unit was developed by the U. S. Army Engineers to supply troops in the field with clean drinking water, regardless of their location or the condition of the water supply, the only prerequisite to a supply of water being to find a stream or reservoir of sufficient volume to supply the demand.

The unit carries a complement of three men, all of whom are qualified to operate the mechanical as well as the chemical aspects of the process. It will handle up to 180,000 gallons of water daily, the amount varying according to the condition of the water to be processed.

Loss-of-Head Determination In Uniformly Tapped Pipes

R. D. Gladding
Soil Conservation Service, U. S. Dept. of Agriculture, Spokane, Wash.

A PROBLEM OF FREQUENT OCCURRENCE is that of determining the loss of head in pipelines having outlets along its length. A particular example would be a lateral sprinkler line used in portable overhead irrigation systems. Others are cooling systems in power plants and sprinkler systems for sewage disposal.

If it is assumed that outlets are evenly spaced along a line and each outlet discharges the same quantity, a simple relationship to the loss of head in the pipe discharging the total quantity at the end may be found.

In Fig. 1 is shown a pipeline AB with outlets spaced s feet apart and each discharging q. Let Q equal the total discharge, or Nq where N is the number of outlets, and L equals Ns.

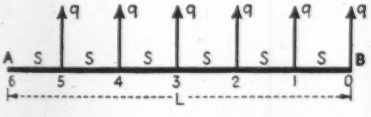


Fig. 1. Pipeline with uniformly tapped outlets.

The general formula for loss-of-head in a given pipe may be shown as $h = K L Q^m$ where K and m are constants depending upon the formula used. Some values of m are as follows:

Formula	m
Darcy	2.0
Williams-Hazen	1.85
Scobey (steel pipe)	1.9

The losses in line AB, Fig. 1, may be summarized as follows:

Pipe section	*Loss-of-Head*
0—1	$K s q^m$
1—2	$K s 2^m q^m$
2—3	$K s 3^m q^m$

Adding,

$$\text{Total loss of head} = K(1^m + 2^m + 3^m + \ldots\ldots N^m)\, s\, q^m$$
$$= K \Sigma N^m\, s\, q^m$$

But, $s = L/N$ and $q = Q/N$

Substituting these values,

$$\text{Loss of head} = h = (\Sigma N^m / N^{m+1})\, K L\, F K L Q^m$$

Application of the formula

In practice it is only necessary to obtain the loss of head as though all the water passed entirely through the pipe and multiply by the value of F as obtained from Fig. 2. It is to be noticed that the factor F depends only upon the number of outlets.

It can be shown by calculus that F becomes $1/(m + 1)$ when N becomes infinite.

If m has a value of 2.0,

$$\Sigma N^m / N^{m+1} \text{ becomes } \frac{2N^3 + 3N^2 + 1}{6\, N^3}$$

Or, $F = 1/3 + 1/2N + 1/6N^3$

As N becomes large, $1/2N$ and $1/6N^3$ approach zero and $F = 1/3$ or $1/m + 1$ which is a check on the method.

As an example, assume a 4-in. pipeline having 10 outlets on 40-ft. spacing, discharging an average of 20 gpm each. The Williams-Hazen formula with $C = 100$ will be used. The value of $m = 1.85$.

The Williams-Hazen tables for a discharge of 200 gpm give a loss of head of 44 ft. per 1,000 ft. of pipe. For 400 ft. of pipe, the loss of head if all the water passed through the pipe would be 0.4 x 44 = 17.6 ft. Using Fig. 2 it is determined that where

$N = 10$ the value of F is 0.405

The loss of head in the pipe with outlets would then be 0.405 x 17.6 = 7.13 ft.

Fig. 2. Loss-of-head chart for determining the factor F when the number of outlets are known.

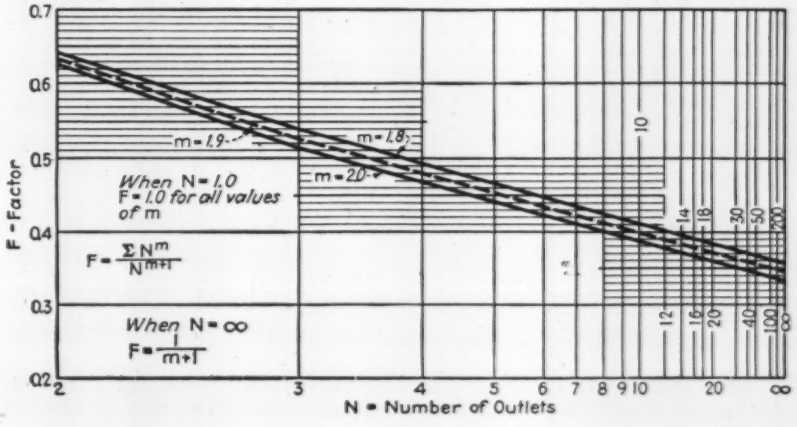


Fig. 1. Steam generation and fertilizer production are byproducts of sludge disposal at Chicago.

World's Largest Sewage Disposal Plant Will Be Doubled in Size

***Contents in Brief**—Work has been started on the enlargement of the Southwest sewage disposal works in Chicago where activated sludge treatment will be provided for a daily flow of 900 mgd. Sludge disposal facilities with an average capacity of 500 tons per day will include provisions for byproduct recovery of fertilizer as well as steam generation.*

EXPANSION of the 400-mgd Southwest disposal works of The Sanitary District of Chicago to provide activated sludge treatment for 900 mgd of sewage, announced in the Oct. 31 issue of *Engineering News-Record*, will double the size of the world's largest sewage treatment works. This will complete the program laid down in 1923 for the complete treatment of all sewage emanating from Chicago and 60 suburban communities.

Under the original plan the 442 square miles of area in the district was divided into four sections. Complete treatment by the activated sludge process was provided for the North Side, Calumet and Southwest areas, where works with total capacity to handle 786 mgd average flow are now in operation. Treatment in the West Side area was to be by Imhoff tanks followed by some form of secondary treatment to be determined later. Thus the West Side works were built with 108 Imhoff tanks having 472 mgd average capacity and the effluent is being discharged to the Chicago ship canal for disposal by dilution.

Complying with the U. S. Supreme Court decree, the diversion of diluting water from Lake Michigan was reduced to 1,500 cfm on Jan. 1, 1939. With the reduced amount of diluting water it has become necessary immediately to provide treatment for the Imhoff effluent from the West Side works, averaging some 400 mgd dry-weather flow. Cross-connections in the sewer systems permit operation of the West Side and Southwest works as a unit, with division of flow to meet operating conditions. Pumping capacity to handle a total flow of 1,550 mgd is installed in the plants.

To provide further purification of the Imhoff effluent several different means were available. An independent board of engineers appointed by PWA to review the plans of the district recommended a study to determine the best means of treating the West Side effluent and funds were provided to cover the construction of part of the experimental plant. These studies, extending over a period of nearly four years, included large-scale tests on six different processes with variations in each.

This resulted in a decision to treat the effluent by aeration with activated sludge. Cost studies showed that it would be more economical to do that by building an addition to the adjoining Southwest works than to build a separate aeration plant at the West Side. Elevation of the discharge from the Imhoff tanks above the water surface of the aeration tanks at Southwest works provides ample head for gravity flow through the 4,100 ft. of conduit as well as meters and gates.

Among the experiments was an ex-

tensive series of tests to determine the results by short-period aeration, which showed that satisfactory treatment could be accomplished by aeration periods considerably less than the five hours used at Southwest works. This indicated a different design of aeration tanks and a different operating rate for the new equipment. Consideration of all design and operating features showed that it would be both desirable and economical to mix the Imhoff effluent with the settled sewage from the Southwest preliminary tanks and treat the combined flow under uniform operation in tanks of the same design as those now in service. Consequently the expansion plan shown in Fig. 2 has been adopted.

When the 400-mgd Southwest treatment work was designed (*ENR*, Aug. 8, 1935, p. 186) the amount of money available did not permit intallation of enough equipment to provide the desired capacity. However, the underground structures were made adequate to handle an ultimate average flow of 1,200 mgd and provisions were made for eventual expansion to that capacity.

The present enlargement to 900 mgd capacity will be made in two stages. The first stage includes the installation of one turbo-blower of 100,000 cfs capacity in the space already available; the construction of 16 final settling tanks; construction of four sludge concentration tanks; enlargement of the sludge disposal plant; and the addition of facilities for handling screenings, scum, grit, coal, and fertilizer. Completion of this work will bring the rated capacity to 500 mgd and provide sludge disposal equipment for the ultimate 900 mgd to be treated.

The second stage will include construction of a conduit from the West Side works with meter vault and gate house; eight aeration tanks similar to those now in use; 24 final settling tanks; installation of two blowers of 100,000 cfm each; and operating galleries, service tunnels, effluent conduits and appurtenances.

Stormwater flow operation

When the flow through Southwest works is increased to 500 mgd, the detention period in the existing primary settling tanks will be reduced from 34 to 27 mins. Operating experience may show that this is not sufficient to give proper solids removal during storm peaks when maximum flow may reach 650 mgd with only 21.3 min. detention. If the settling period is not enough it may be increased by building four more primary tanks which will increase the detention at maximum flow to 28.3 min. Since the present tanks accomplish an average of about 50 per cent removal of suspended solids in 30 to 35 min. detention, construction of additional tanks is held in abeyance.

The general plan of The Sanitary District is to intercept and treat storm runoff equivalent to 50 per cent of normal dry-weather sewage flow. That plan will be followed in the operation of the combined West Side-Southwest works. All flow (150 per cent of dry weather) will receive primary sedimentation but only that part equivalent to 115 per cent of dry-weather will receive complete treatment.

Excess solids will be stored for short periods in the aeration system by increasing the concentration of the mixed liquor as high as 4,500 ppm to smooth out the storm maxima and reduce the amount of sludge disposal equipment that would be required to remove solids as fast as they are received. The mixing of Imhoff effluent with the settled sewage of the Southwest plant will also smooth out the large variations in strength of the latter due to periodic discharge of highly concentrated wastes from stockyards and packinghouses.

Second stage construction

Principal items in the second stage of construction are the eight 4-pass, spiral-flow tanks 434 ft. long by 134.75 ft. wide with water depth 15 ft. over the diffuser plates, and the 24 final settling tanks. Also the existing pump and blower house and sludge disposal building will be extended to receive the two 100,000-cfm blowers and three complete sludge drying and steam generating units including a fertilizer handling and loading plant. These additions will provide capacity to dry 500 tons of sludge, daily average, with reserve capacity for 165 tons more for peak loads. Steam generating capacity will be 590,000 lb. of steam per hour at 425 lb. pressure and 735 deg. final temperature.

Design and construction of this work are by the engineering department of The Sanitary District of Chicago under the direction of Wm. H. Trinkaus, chief engineer, H. P. Ramey, assistant chief engineer, L. C. Whittemore, engineer of treatment plant design, Wm. A. Dundas, engineer of mechanical design and L. B. Barker, engineer of construction.

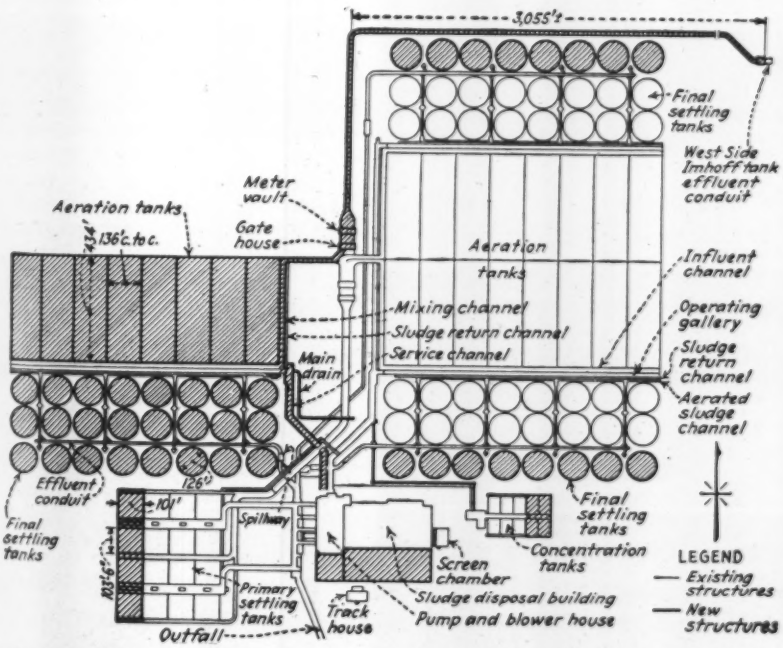
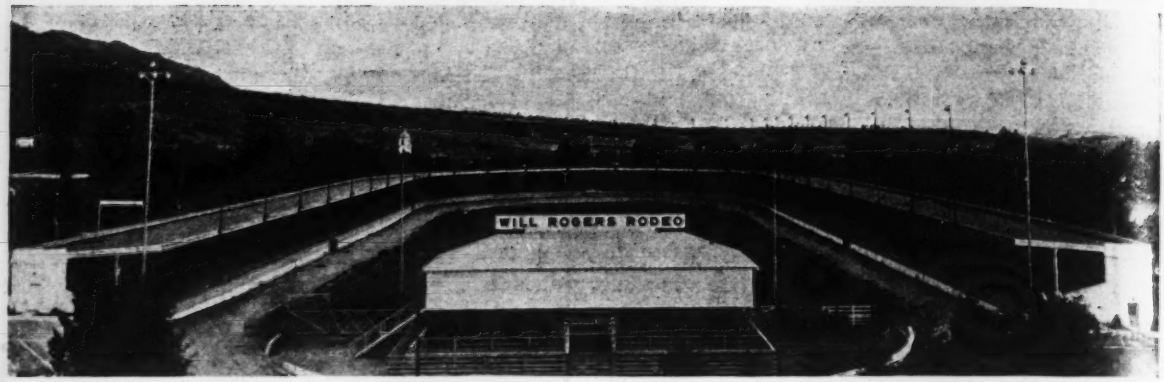


Fig. 2. Additions to be made at the Southwest disposal work (indicated by cross-hatching) will increase its capacity from 400 to 900 mgd, making it the world's largest activated sludge plant.



Will Rogers stadium in Colorado Springs, next to the Broadmoor Hotel, is located in picturesque surroundings.

An All-Welded Steel Stadium

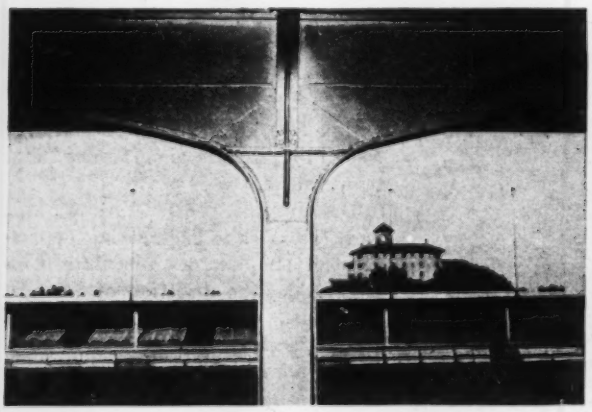
J. H. Sanford and O. O. Phillips
Consulting Engineers, Colorado Springs, Colo.

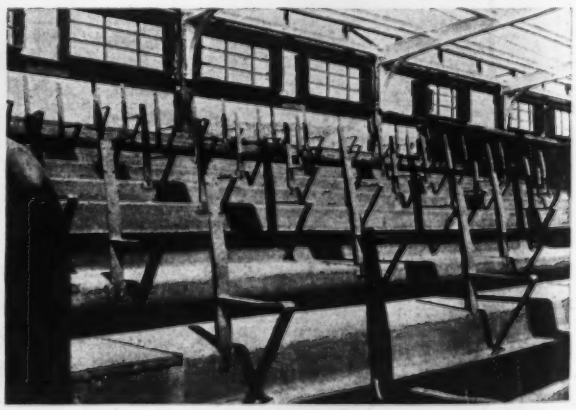
***Contents in Brief**—The Will Rogers Stadium in Colorado Springs provides all the conveniences and facilities necessary to night and day sports. Continuous rigid-frame roof supports, steel-plate seating deck, steel fascia and ramp walls are all electrically welded.*

CONCEIVED AND BUILT by the late Spencer Penrose in memory of the cowboy humorist, the Will Rogers Stadium was designed to afford every comfort and convenience which should be incorporated in a modern rodeo plant. Providing seating capacity, all under cover, for 10,000 spectators, the structure, built entirely of welded steel, is located near the Broadmoor Hotel, Colorado Springs, Colo., in the picturesque surroundings of the Pike's Peak region. Although designed primarily as a rodeo field, provision was made to accommodate a number of other outdoor sports such as football, track, horse shows, etc.

The stadium is a horseshoe, 305 ft. wide and 560 ft. long. The bandstand, stock pens and chutes are in the open end and pick-up pens at the opposite end, thus giving the stock a run of the full length of the field in view of the spectators. A removable fence encloses the rodeo field, allowing space between the fence and the stands for a quarter-mile race track. The entire field and race track is brilliantly floodlighted for night performances.

Under the deck of the stadium are thirty bedrooms, each with private bath and individual outside entrance. The bedrooms are for the use of the contestants or to serve as an annex to the hotel. The balance of the space under the seating deck is utilized for equipment storage, ticket compartments, offices, dressing rooms, public toilets, sandwich shop and other concessions. Surrounding the stadium are wide concrete sidewalks and the grounds are beautifully landscaped which enhance the





Split beam construction at the top of column under a rigid frame girder supporting the cantilever roof. Right: Steel brackets supporting the seats are welded to the steel plate deck.

well-appointed architectural design.

Resting on concrete foundations, the stadium proper is an all-steel structural frame, the principal features of which are the continuous rigid-frame roof support, steel plate seating deck, steel fascia and ramp walls. The roof support on the front line of columns is an electrically-welded, continuous rigid frame of the beam-arch type which is designed to act integrally with the columns. The basic element of the beam arch is an 18-in. wide flange beam, the web of which is split horizontally at each end and the bottom flange then bent down to a predetermined curve to provide the necessary increase in depth at points of maximum moment. The resulting V-shape web openings are filled with added plate material, electrically welded, to form a clean-cut solid web. The tops of the columns were likewise formed to the required curve, and the two field-welded in place to form one unit. Architecturally the arches present a pleasing and graceful profile and fulfill the demand for esthetic treatment of modern structures.

Permanent seats are provided with back rests and are supported by steel brackets which are welded to the $\frac{1}{4}$-in. steel-plate deck. The deck construction utilizes L-shaped bent-plate units, each making up a tread and riser. Alternating lugs on the abutting edges provide an effective interlocking of the units and hold them in place during erection, after which all seams are welded water-tight. The steel-plate deck is self-supporting between the rafter beams, which are spaced on $14\frac{1}{3}$-ft. centers. The side walls of the ramps are of steel plate as is the fascia in front of the boxes.

Aisles are provided with an asphalt-impregnated fiber board walking surface, and the floors of the boxes, main aisles and ramp runways are constructed of a steel grillage filled with concrete.

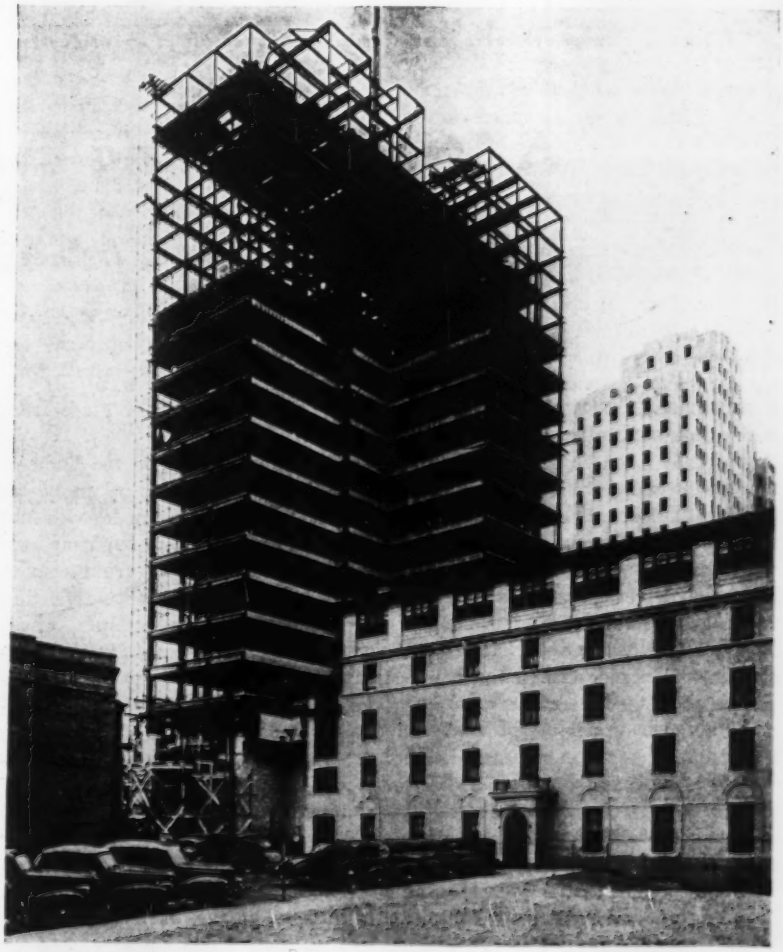
The use of steel deck plates, steel ramp walls and concrete aisles and runways results in a structure which is economical to maintain and easy to keep clean.

The rear walls are structural steel frames covered with fire-resistant magnesite-base fiberboard insulating slabs and stucco, thus providing protection for the rooms under the seat deck. Other walls and ceilings of the rooms are also insulated and plastered. The roof is an asphaltic composition laid on 2-in. sheathing and fiberboard insulation.

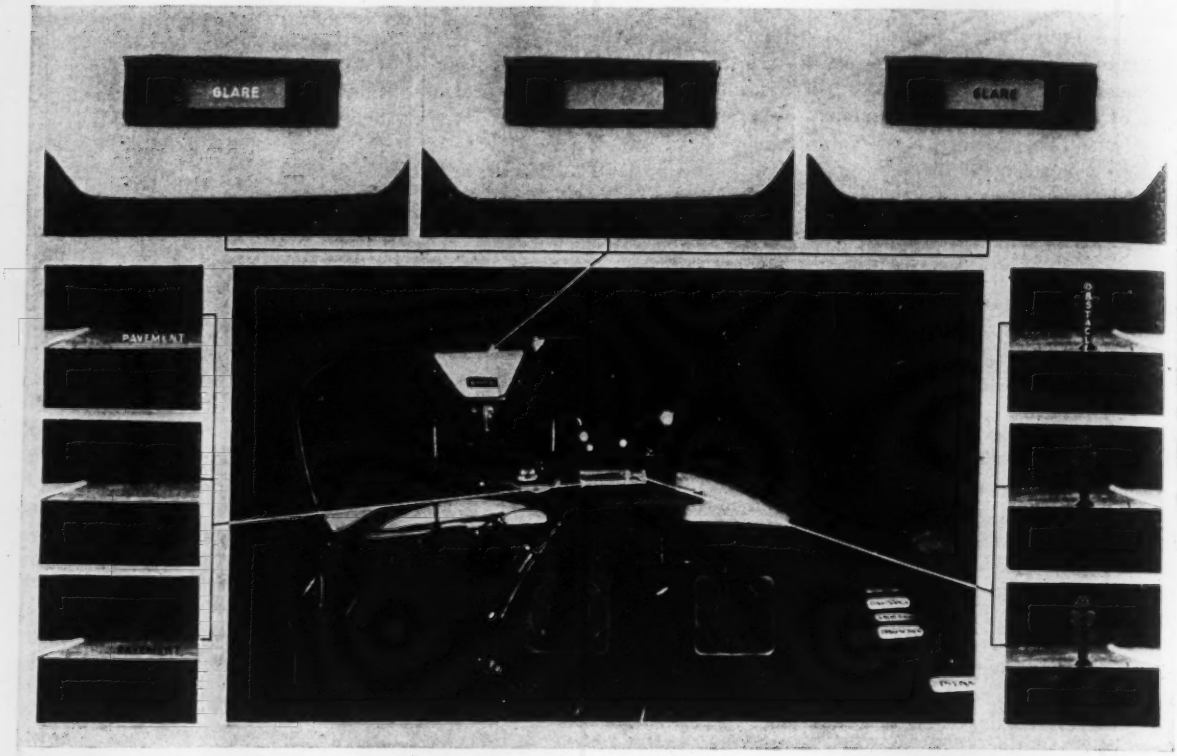
In the building of the Will Rogers Stadium M. J. Strong was construction superintendent for the owner. Design and construction were in charge of Charles E. Thomas, architect, and Sanford and Phillips, consulting engineers, Colorado Springs, Colo. The steel work was jointly fabricated by the Virginia Bridge Co. and the American Bridge Co. with the latter company doing the erecting work.

Building Contains Four Miles of Welds

TALLEST ALL-WELDED BUILDING in the world, the 23-story Nurses Home in the Medical Center of Jersey City, N. J., contains more than 18,000 lin. ft. of fillet welds and about 1,900 lin. ft. of butt welds, a total of nearly four miles of welds in a steel tonnage of 2,200. Only 63 tons of this amount represent connection material. The fillet welds range from $\frac{1}{4}$ to $\frac{3}{8}$ in. in size, and the butt welds from $\frac{5}{16}$ to $\frac{15}{16}$ in. Windbracing connections are unusually heavy, top connection straps being as thick as $\frac{7}{8}$-in. plates butt-welded to the columns, and in some instances are strong enough to develop the full bending strength of the connected beams. Together with the adjacent 16-story Hague Memorial Hospital, the building includes 3,300 tons of structural steel. Lehigh Structural Steel Co. fabricated the framework and Essex Steel Erectors erected it for Christian H. Ziegler, Jersey City, architect, Lars I. Moe of Jersey City, structural engineer and Gilbert D. Fish, consulting welding engineer.



Building has 18,000 lin. ft. of fillet welds and 1,900 lin. ft. of butt welds.



Schematic view of evaluater for determining effectiveness of lighting system and visibility of different pavements.

A Street Lighting Evaluator

Kirk M. Reid and E. J. Channon
Nela Park Engineering Department
General Electric Co., Cleveland, O.

A NOVEL NEW INSTRUMENT recently developed determines the visibility provided by a road lighting system or the effect on visibility of different pavement surfaces. This instrument is mounted on a car and it is operated and the readings are taken, directly from the driver's seat. The accompanying illustration is a schematic view of the installation and its operation. Briefly described the installation consists of (1) a pavement bed mounted over the hood of the car, (2) a glare integrator mounted above the windshield, and (3) a control box on the seat beside the operator. It measures three things, which collectively determine the effectiveness of a lighting installation: (1) the brightness of the pavement; (2) the brightness of representative obstacles on and near the roadway, and (3) the veiling brightness of glare from sources in the field of view.

The pavement bed on the hood of the car is fitted with a pavement sample which is similar in tone and texture to the street or highway surface. Selection is made from a series of samples with different reflection characteristics. For example, where consideration is being given to resurfacing a roadway by incorporating slag, granite chips, etc., to increase reflectance, the effect on the net visibility can be determined beforehand, using a pavement sample simulating the proposed material.

Measuring pavement brightness

The next step is to measure the brightness of the pavement sample. This is done from the driver's position by a photometer housed in the pavement bed. The pavement sample becomes the test field of this photometer. The comparison element is the horizontal word PAVEMENT which is printed on clear glass in white pigment with opaque back and is controllably illuminated from beneath the pavement bed. By varying the current passing through the lamp, which lights the horizontal word PAVEMENT, the lettering becomes lighter or darker than the pavement sample. The point where the word disappears is the point of photometric balance.

In a like manner, the operator measures the brightness of a pedestrian obstacle of typical reflection, which is placed on the pavement strip. He views the obstacle adjacent to and between the word OBSTACLE, lettered on the glass plate.

Again the point of balance, or brightness of the obstacle, is reached when the word fades from view.

The third measurement has to do with glare produced by sources in the field of view. This is made possible by the glare integrator which is placed above the windshield. The element of the integrator which enables this measurement to be made is the transparent diffuser. It is so designed that it integrates the brightness of the various sources, no matter where located within the field of view, in direct proportion to the interference with seeing. The diffuser is mounted at the end of the glare integrator which comprises the upper part of the housing and with the aid of a level is positioned perpendicular to the street surface. A photometer combined with a periscope permits the readings of glare to be made from the driver's seat. The operator views the comparison element, the word GLARE, against the field of the transparent diffuser. The reading is recorded at the point where the word disappears.

These readings may be made excluding the effect of automobile headlights. This is an important factor because it enables measurements to be made any time after dark, providing rain and snow are not falling, without having to wait until late at night when traffic is small. The measured results of pavement brightness, obstacle brightness and veiling brightness are punched on a record card for permanent record. This card, which is positioned in a holder attached to the photometric control box, also includes a graphical means of computing the visibility from the brightness values measured. The device was developed by the authors with the assistance of C. B. Buck, Jr.

Water for a Desert Town

Herbert T. Bruce
Flushing, New York

Contents in Brief—A Chinaman started the idea of purveying water in the frontier town of Elko, Nev., and from there on American enterprise improved upon the idea. Today a $330,000 investment in a ground water supply is practically amortized, and there is no general water tax in this city of 4,000, fifth largest in the state.

ELKO, NEVADA, is indebted to a Chinaman for its water system. He tapped the springs of Elko Mountain, which rises startlingly from the city's edge, and trucked it into the then wild and woolly cattle town. The ranchers had to drink water—once in a while—so the Chinaman's commodity was sought after since it was simpler to buy it from him than to find a spring or dig a well. But the town grew; ranchers and miners spread out through the Ruby Mountain valleys and made Elko a supply base. It became the county seat, and then a city of some four thousand—the fifth largest city in Nevada. By now, however, it needed water for other than its thirst-quenching value, and the Humboldt River, which flows through the city, is dry seven to eight months of the year. So the city fathers ran the Oriental out of town and took over his source of supply.

To see Elko from the air, and to ride on horseback over the sandy outskirts of the city as the writer did, one would wonder where the water comes from and where it goes. There is no river which can be tapped, other than the more-often-than-not dry Humboldt; there are no mountain streams from which it can draw. The soil is absorbent to the depth of rock; no Nevadian river ever flows out of the state, there being a natural "sink hole" into which rivers ultimately flow and disappear beneath the earth's surface. Therefore, the city's water problem was a problem indeed when population growth occurred. As ranchers moved east, west, north and south into the Ruby valleys and miners moved west and north Elko became a headquarters for legal advice in connection with mining claims, the forming of corporations and the settling of water claims, land boundaries and other rights of the ranchers.

Location map of the city of Elko.

Wooden pipes used

Water supply facilities were crude in the 1870's, and cruder still in Nevada. The first step taken by those authorized to build a water system was to make and lay pipelines. The pipes were square pieces of timber with a hole bored through the center, similar to the wooden pipes laid by The Manhattan Company in New York in the late eighteenth century. The next step was to locate, in this seemingly dry prairie, springs fed from the canyons in the mountains beyond. It was soon learned that wherever the quaking asp grew (a tree very similar to the white birch of the eastern mountains), water was to be found. Then came the problem of tunneling from these springs to central city distributing points. The wealthier of the city's inhabitants financed those operations as a private venture.

It was soon found that the tunnels robbed each other, thus depriving one section of the town of its supply. As an alternative, sump holes were dug in the dry bed of the Humboldt. But this was found to be unsatisfactory, because the water was not sufficiently pure for domestic purposes. Many of the city's inhabitants, therefore, enthusiastically accepted weekend invitations to neighboring mines and ranches where good water was to be had, and they returned with kegs filled with a week's supply for drink-

Elko, Nev., the fifth largest city in the state, with a population of 4,000.

Outskirts of the city do not look promising for water supply purposes.

ing. In 1923, however, when the city bought the water supply franchise from its private owners, the Elko Municipal Water Works was inaugurated and plans were made for what was then thought to be an adequate and permanent system.

Wells were sunk to a depth of about 175 ft. by means of an old oil drill. These wells were capable of producing 300 gpm, which was thought sufficient to satisfy demands for years ahead. But it wasn't. So Elko sank more wells and increased its supply to 397 gpm (562,000 gpd). This was in 1927. In 1937, the capacity was increased to 821,000 gpd.

Coming down the winding highway on Elko Mountain from the prairie grazing grounds between Elko Mountain and the Ruby Mountain Range, one looks across the city and wonders how any kind of vegetation prospers. Sagebrush hills surround the hollow in which the city is built. One half of the hill opposite the mountain highway is residential; the other half is given over to a golf course, the Fair Grounds and, on the very crest of the hill, are two water storage tanks holding a half day's supply. Four pump installations located in different parts of the city force water into the upper tank which overflows into the lower. One of the pumps has a capacity of 100 gpm; the other three have capacities of 520, 670 and 300 gpm, respectively. Deep-well, diesel-driven pumps are used, an auxiliary gasoline motor starting the diesel engine and also serving as a standby unit.

Wells must be dug on deltas from side canyons. Water can usually be struck at a depth of 17 to 30 ft. But this water is generally drainage water and is unfit for drinking. Good water is reached only by drilling through strata of clay and rock to a depth of 70 to 100 ft.

Nevada is a throw-back to the western romance of the last century; therefore, its cities with their up-to-date public works facilities seem incongruous by comparison. Elko, with its modern water supply takes justifiable pride in this accomplishment which was made possible by a bond issue totaling $330,000. This debt is now nearly amortized.

The waterworks are self-supporting yet the city levies no general water tax; only stores, garages and similar commercial buildings are charged for service. The minimum water rate is 30c. for the first thousand gallons and 10c. for each additional thousand gallons in excess.

Considering that the streets are piped for fire service and that the water department undertakes other municipal operations, such as swimming pools, it is exceptional that no general water tax is levied. Elko is not a wealthy community in the sense that it is not supported by large manufacturing plants or industries, taxes from which could be used to defray a variety of municipal expenses.

Robert Kinne, Elko's Commissioner of Water Works, wound up his guiding tour of the system by taking the writer to the "Hot Hole" swimming pool project. Here is a natural pool of some fifty feet in diameter fed by boiling hot springs. The water contains mineral properties which are health-giving to those afflicted with constitutional weaknesses. Sometime ago private interests built a concrete swimming pool fed by this water, and admission charges were made. Waterworks officials, however, recognizing the value of these baths to everyone, recommended that the city buy the property. This was done and now a normal-temperature pool accompanies the hot-water pool, and a shallow pool has been built for children. It has become a seasonal resort for young and old during the warm months of the year.

Thus has prairie water been found and distributed for the public good. A Chinaman started the idea in Elko, and American enterprise improved upon it.

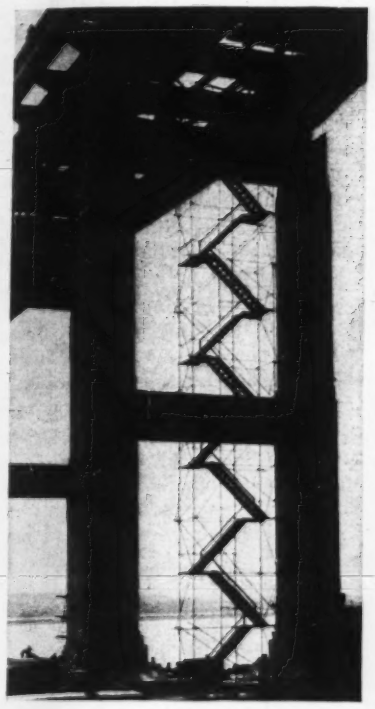
• • •

Sensitive Level Vials For Accurate Surveys

Accurate survey work requires sensitive level vials, according to Louis H. Berger, of C. L. Berger & Sons, Boston, builders of surveying instruments. A proper relation between vernier readings and level sensitivity is important. The telescope level for ordinary transits, wye and dumpy levels may have a sensitivity of either 30 sec. or 20 sec. per 2-mm. division. The plate level should have a sensitivity of at least 60 sec. per 2-mm. division when the horizontal verniers of the transit read to 1 min. or less. Plate verniers reading to 10 sec., as in a 7- or 8-in. theodolite, require plate level vials with a sensitivity of 45 sec. per division; the telescope vial in this case may be as sensitive as 20 sec. per division.

From Field and Office

Working problems and time-saving methods for engineers and contractors



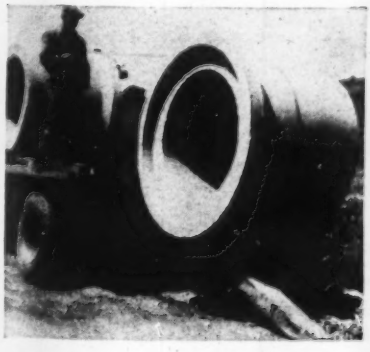
Edison Bridge tubular scaffold stairway.

Tubular Stairway Scaffold

Tall tubular scaffold carries stairway providing safe access to the high level deck of the Edison Bridge over the Raritan River at Perth Amboy, N. J. Bethlehem Steel Co., superstructure contractor, erected the scaffold.

Easy Does It

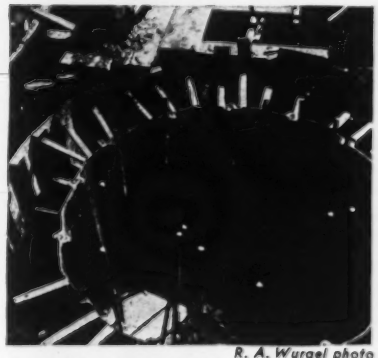
The power takeoff operated cable on the truck lets the 48-in. extra heavy concrete pipe roll easily down the skids at Wheeling, W. Va., airport.



The truck driver and the man with the bar, who starts the pipe rolling, form the entire crew for handling the 6,000-lb. pieces.

Shop Bent Piling for Cofferdam

Steel sheetpiling, 15 in. wide and $\frac{3}{8}$ in. thick is bent 20 deg. in the center of each sheet and formed into a 5-ft. 6-in.-diameter circle and driven 35 to 40 ft. to rock for Department of Sanitation dumping board along the new East River Drive in New York. The sheetpiling makes the cofferdam and is cleaned out to rock, then filled with concrete to form a foundation pier. P. T. Cox of New York is the contractor.



R. A. Wurgel photo

Sheetpiling for 5-ft. 6-in. cofferdam.

Unusual Lamp Shades

Ornamental light standards, as here shown, have been made in the shops at Grand Coulee Dam for use in a small park at the foot of Administration Ave. in the permanent town of "Coulee Dam." The vertical rods are 1-in. granite cores made in test borings at the damsite. These are set in pipe caps, and the entire assemblies were made up from standard parts or scrap material in the shop. One disadvantage is that visitors to the park sometimes break out the granite cores.

Ornamental lamp gets a decorative effect from a group of 1-in. drill cores.

Trailers for Job Delivery

Although delivery by motor truck is the usual program for materials that do not reach Friant Dam by rail, there is always some movement of material from place to place about the job for which it is undesirable to keep a motor truck tied up during the loading process or while awaiting opportunity to unload. For this purpose trailers have been found very useful and a considerable number are in service. For the most part, they are built on the job from old automobile bodies. The steering bar on the front wheels is attached to a tongue so that the trailer can be conveniently moved about by hand when empty or can be towed.

Automobile trailer used for quick job delivery.

Notes on Sewage Disposal

A monthly summary of current developments conducted by Willem Rudolfs

CHLORINE AND POLIOMYELITIS —Although the poliomyelitis virus was found in human feces as early as 1912, it is only during the last two years that water and sewage have been linked more closely with epidemics of infantile paralysis. Following the detection of the virus in sewage in the Charleston and Detroit epidemics of 1939, general interest was aroused. The presence of the virus in human intestinal discharges led some French investigators (1913) to study the effect of chlorination. A concentration of 4 ppm destroyed the virus in a cloudy tap water emulsion, while 0.4 ppm was effective with clarified preparations.

Recently Kempf and Soule (*Proc. Soc. Experimental Biology and Medicine, June 1940, p. 431*) studied the effect of chlorine in city water at Ann Arbor, Mich., using for this purpose a suspension of the virus. They found that chlorine in concentrations of 0.5 ppm did not inactivate the virus in 1½ hr. The authors conclude that the usual municipal practice of retaining a residual chlorine content of 0.1 to 0.2 ppm is insufficient to inactivate the virus. A higher chlorine concentration and longer contact time are deemed necessary.

"The possibility that drinking water, adequately chlorinated according to accepted standards, may be a factor in the epidemology of poliomylclitis must be recognized as a result of these findings." The authors also call attention to shortcomings in the protection of swimming pool waters, since carriers may discharge the virus from the intestinal tract or naso-pharynx and the chlorine content is apt to drop significantly during peak bathing loads.

BETTER GRASS —Lawn irrigation with the final effluent from the sewage treatment plant is being practised at the State Tuberculosis Sanitorium in South Dakota. As explained by the health authorities in a recent bulletin, the sewage is chlorinated to destroy pathogenic bacteria, and it is reported that no tuberculosis organisms have been discovered in the chlorinated sewage. Further tests will be made to verify the preliminary investigations. In addition to destroying pathogenic organisms, chlorination has been found effective for odor control.

BIG BLOWERS —What are claimed to be the three largest turbine-driven blowers for sewage aeration are being built by the Allis-Chalmers Mfg. Co. for The Chicago Sanitary District at a cost of $550,000. The blowers will each have a capacity of 115,000 cu.ft. a minute at 8.5 lb. gage and each will have its own condensing unit, connecting pipe and air check valves.

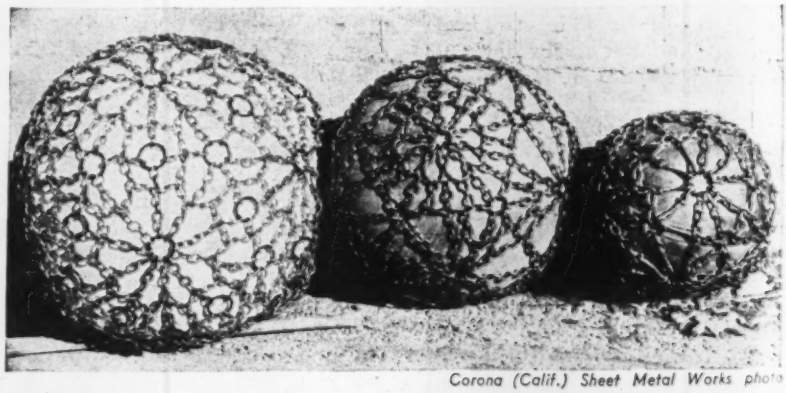
PIPE CLEANING—The use of rubber balls covered with a chain netting made to roll along in a sewer pipe is a novel and simple way to clean grease from the sides, stir up sand and sludge from the bottom of the pipe or remove obstructions such as caused by excessive quantities of garbage. The chain-enclosed ball, available in various sizes, is inflated with air until it fits the pipe fairly snugly. The ball is then permitted to roll along the top of the pipe during which it carries debris or scum ahead to the next manhole. After the major obstructions are removed the ball is filled with water and permitted to roll along the bottom of the pipe. If a ball gets stuck it can be either pushed out or deflated by puncturing.

SEWAGE STRENGTH —Sewage with a biochemical oxygen demand and suspended solids concentration of about 200 ppm is considered average for domestic sewage. In certain sections of the country these average figures may require revision. To assist consulting engineers in planning adequate sewage treatment plants, a study of raw sewage strengths, based on weighted samples, was made at 73 plants by the Texas bureau of sanitary engineering of the State Department of Health. This study is expected to be of assistance to plant superintendents in determining whether their treatment plants are receiving ordinary strength wastes. If the strength is greatly in excess of the average figures for similar towns the reason should be sought by studying contributed industrial wastes.

The average results from analyses of raw sewage obtained at 73 plants were:

Biochemical oxygen demand	356 ppm
Suspended solids	363 ppm
Oxygen consumed	132 ppm
Ammonia nitrogen	38 ppm
Chlorides	284 ppm
Alkalinity	442 ppm

Alkalinity and chloride figures are affected by the hard waters. Surprisingly, the strength of raw sewages was higher than expected. Of the 73 sewages, 50 per cent had B.O.D. values from 300 to 500 ppm, 40 per cent below 300, and 10 per cent over 500 ppm. It would be of considerable value for general use, if the water consumption figures for these towns were available.



Corona (Calif.) Sheet Metal Works photo

Rubber balls with chains are the latest device for sewer pipe cleaning. Filled with air, they scrape the top of the pipe; filled with water they clean the bottom.

Book Notes and Reviews

The month's addition to the engineer's reading and reference list

Airport Planning

MASTER PLAN OF AIRPORTS for the Los Angeles County Regional Planning District—Published by the Regional Planning Commission, Los Angeles, Calif. Limited edition.

While this report relates primarily to the Los Angeles region, the treatment of the subject is sufficiently general to make the book useful to those attempting to lay down a logical airport program for other metropolitan regions. All factors bearing on the design and location of the several types of airports needed for adequate service to a large metropolitan district are discussed.

Line Clearances

TREE CLEARANCE FOR OVERHEAD LINES—By George D. Blair. 238 pp. Published by Michigan Electrical Publication, Chicago, Ill. Price $3.75.

Municipal engineers who are troubled by the competition between power lines and trees for space along their streets will find much of value in this book. It treats of such subjects as proper care of trees, tree heights and growth rates, trimming, standard clearances and regulations affecting clearances. — *Reviewed by* KARL DRESSEL, *assistant professor of forestry, Michigan State College.*

A Militant Engineer

PANAME TO VERDUN—By Col. Philippe Bunau-Varilla. 272 pp. Published by Dorrance and Co., Philadelphia. Price $2.50.

Philippe Bunau-Varilla was a fighter and a promoter first and an engineer second, but until the World War most of his fighting was done by means of the spoken and written word. His greatest battle was to salvage something from the wreckage of the collapse of the French efforts to build a canal at Panama. That ended with the signing of the Hay-Bunau-Varilla Treaty which assured American support for the newly formed Republic of Panama.

Bunau-Varilla himself played a part in the closing years of the French construction. That period he treats only in a sketchy but pungent way. The greater part of the book is devoted to his efforts to get the United States to take over the canal construction. From the historical angle, many will question the weight that Bunau-Varilla gives to his own actions and opinions, but none will find his somewhat egotistic account of those days dull reading.

Mathematics for Engineers

MATHEMATICAL METHODS IN ENGINEERING—By Theodore V. Karman and Maurice A. Biot. 505 pp. Published by McGraw-Hill Book Co., New York and London. Price $4.

The preface to the volume here reviewed answers pretty clearly questions inevitably preceding the writing of a book—why is the book written, for whom, how is the material to be "sold" to the reader. These authors write to give "a better understanding of the potentialities" of mathematics, they have chosen to explain by use of specific illustration more than by general exposition, they write for mathematicians, engineering instructors, students, and practicing engineers.

The examples chosen are chiefly from structural engineering, from oscillatory problems, and from transient phenomena. Many—probably too many—mathematical devices are presented. The writers have avoided the rather elusive problems of defining "mathematics" and "engineer."

Two chapters are devoted to structural engineering. One of these deals with applications of differential equations to such problems as the shape of a loaded string, beam deflections, some problems of buckling, the theory of stiffened suspension bridges, and to some problems of vibration. The other chapter introduces Fourier series; many structural engineers would expect to find there a discussion of slabs. A few graphical methods, including the familiar procedure for the construction of moment diagrams, are explained.

A closing chapter, called "Words and Phrases," may interest many engineers who either do not use higher mathematics or make it up as they go along and therefore do not know very well the diction of the field.

It may seem needless to summarize that many will find the book interesting; the scholarly eminence embodied in the combined careers of the authors almost guarantees that result.—*Reviewed by* HARDY CROSS, *professor of civil engineering, Yale University.*

Railroad Transportation

THE MODERN RAILWAY—By Julius H. Parmelee. 730 pp. Published by Longmans, Green & Co., New York, London and Toronto. Price $4.

Rail transportation is viewed broadly in this text by the director of the bureau of railway economics of the Association of American Railroads. Also discussed are water and highway transport because each has a direct bearing on the rail transport problem of the country.

Engineers who wish to bring themselves up to date on such matters as valuation of railroad property, consolidation, and the effect of the Motor Carrier Act of 1935 on the nation's transportation system will find these subjects and a variety of other concisely treated in this book.

Drawing for Engineers

TECHNICAL DRAFTING—By Charles E. Schumann. 793 pp. Published by Harper & Brothers Publishers, New York and London. Price $3.50.

TECHNICAL DRAWING—Second Edition. By F. E. Giesecke, A. Mitchell and H. C. Spencer. Published by The Macmillan Co., New York. Price $3.

These new texts on drawing for engineers and architects are compiled primarily for teaching, but each is also designed to meet the requirements of the practicing engineer in the way of standard methods, symbols and other elements of technical drawing.

"Technical Drawing" is a revised and amplified edition of a text that was published in 1933. Its emphasis is on mechanical drawing and a considerable part of the text is given over to page plates giving a wide variety of such work. Large supplementary plates by the same authors are published under the title "Technical Drawing Problems."

The Schumann text is new. It is

divided into two parts, one treating of the elements of engineering drafting and the second of professional drafting. A new departure in presentation of the theory of engineering drafting is omission of the planes of projection. However, for those who prefer to use the conventional method in teaching, the author has included a chapter on that method.

Over 60 pages of space in the Schumann text are devoted to the drawing of welder details. Included are the standard welding symbols approved by the American Welding Society. Both texts have a chapter on engineering graphs and both include all standard symbols for drawings.

Keeping Cost Records

COST ACCOUNTING — By John C. Blocker. 705 pp. Published by McGraw-Hill Book Co., New York and London. Price, $4.

This book is offered as a text for students of accounting. One chapter is devoted to cost accounting for government organizations. The importance of budgeting and its relation to cost accounting is forcefully brought out, and this material should be useful to cost accountants in the construction industry.

Distribution of overheads; production order cost systems; social security tax accounting; the Robinson-Patman Act; control of distribution costs; material control systems, all come in for attention by the author. —*Reviewed by* PAUL FAHEY, *chief, construction accounting division, Tennessee Valley Authority.*

Miscellaneous Notes on Booklets and Reprints

WATER CONSUMPTION in hotels and other statistics relating to uses and conservation of water are available in a book published by the American Hotel Association, 221 W. 57th St., New York, N. Y., price, $1.

CEMENT STANDARD, including specifications, chemical analysis and physical tests are given in a pamphlet entitled "A.S.T.M. Standards on Cement," published by the American Society for Testing Materials, 260 South Broad St., Philadelphia, Pa. (Price, $1). The pamphlet includes the recently adopted tentative specifications covering five types of portland cement and the newly standardized test for autoclave expansion. Also included are specifications for high-early-strength portland, natural and masonry cement, and methods for conducting tests on cement and cement mortars.

MOTOR VEHICLE SIZE AND WEIGHT LIMITATIONS is the subject of three preliminary reports prepared by the Interstate Commerce Commission, under the provisions of the Motor Carrier Act of 1935. Report No. 1 covers state limitations on sizes and weights of motor vehicles, with an analysis of the statutes of each state. Report No. 2 covers road facilities and vehicles used in highway transport, including data as to vehicle characteristics drawn from the state-wide highway planning surveys, and Report No. 3 covers sizes and weights of motor vehicles in relation to highway safety. Copies of these reports can be obtained from the National Highway Users Conference, National Press Building, Washington, D. C. (Price, $1).

New Engineering Books

WARTIME CONTROL OF PRICES—By Charles O. Hardy. 216 pp. Published by The Brookings Institution, 722 Jackson Place, Washington, D. C. Price $1.

WORKING STRESSES—By Joseph Marin. 41 pp. Published by Rutgers University Press, New Brunswick, N. J. Price $1.

SIMPLE BLUEPRINT READING with particular reference to Welding and Welding Symbols. 138 pp. Prepared and published by The Lincoln Electric Co., Cleveland, Ohio. Price 50c. in the U. S. A. 75c. elsewhere.

LAND DRAINAGE AND FLOOD PROTECTION—By Bernard A. Etcheverry. 327 pp. Published by Stanford University Press, Stanford University, Calif. Price $3.

VENEZUELA—By Henry J. Allen. 289 pp. Published by Doubleday, Doran & Co., New York. Price $3.50.

SEWAGE TREATMENT WORKS—By C. E. Keefer. 673 pp. Published by McGraw-Hill Book Co., New York and London. Price $6.

SEWERAGE AND SEWAGE TREATMENT—By Harold E. Babbitt. Fifth Edition. 648 pp. Published by John Wiley & Sons, Inc., New York; Chapman & Hall, Ltd., London. Price [illegible]

EXPLORATION GEOPHYSICS—By J. Jakosky. 786 pp. Times-Mirror Press, Los Angeles, Calif. Price $8.

CONCRETE DESIGN AND CONSTRUCTION—By W. Herbert Gibson and Walter Loring Webb. Second Edition. Revised by W. Herbert Gibson. 568 pp. Published by the American Technical Society, Chicago. Price $4.75.

BOUNDARY PROBLEMS AND DEVELOPMENT PROJECTS—By D. C. W. Ricketts. 128 pp. and 140 plates. Published by the Pelican Publishing Co., 339 Carondelet St., New Orleans, La. Price $4.

Reports and Pamphlets

WHAT THE BUSINESS MAN SHOULD KNOW OF THE LABOR LAW AND ITS ADMINISTRATION—By J. Raymond Tiffany and Benjamin Werne. May be obtained from Hon. J. Raymond Tiffany, 25 West 43rd St., New York, on the following basis: 1-50 copies, 50c. each; 51 to 250, 40c. each; and 251 and over, 25c. each.

DIGEST OF STATE AND FEDERAL LABOR LEGISLATION—Bulletin No. 40, U. S. Dept. of Labor, Div. of Labor Standards. May be obtained from the Supt. of Documents, Washington, D. C. Price 10c.

FACTORS AFFECTING THE COST OF REFUSE COLLECTION, REFUSE COLLECTION METHODS, REFUSE COLLECTION EQUIPMENT — Bulletins Nos. 10, 11 and 12. Prepared by the American Public Works Association, 1313 East 60th St., Chicago, Ill. Price 50c. each.

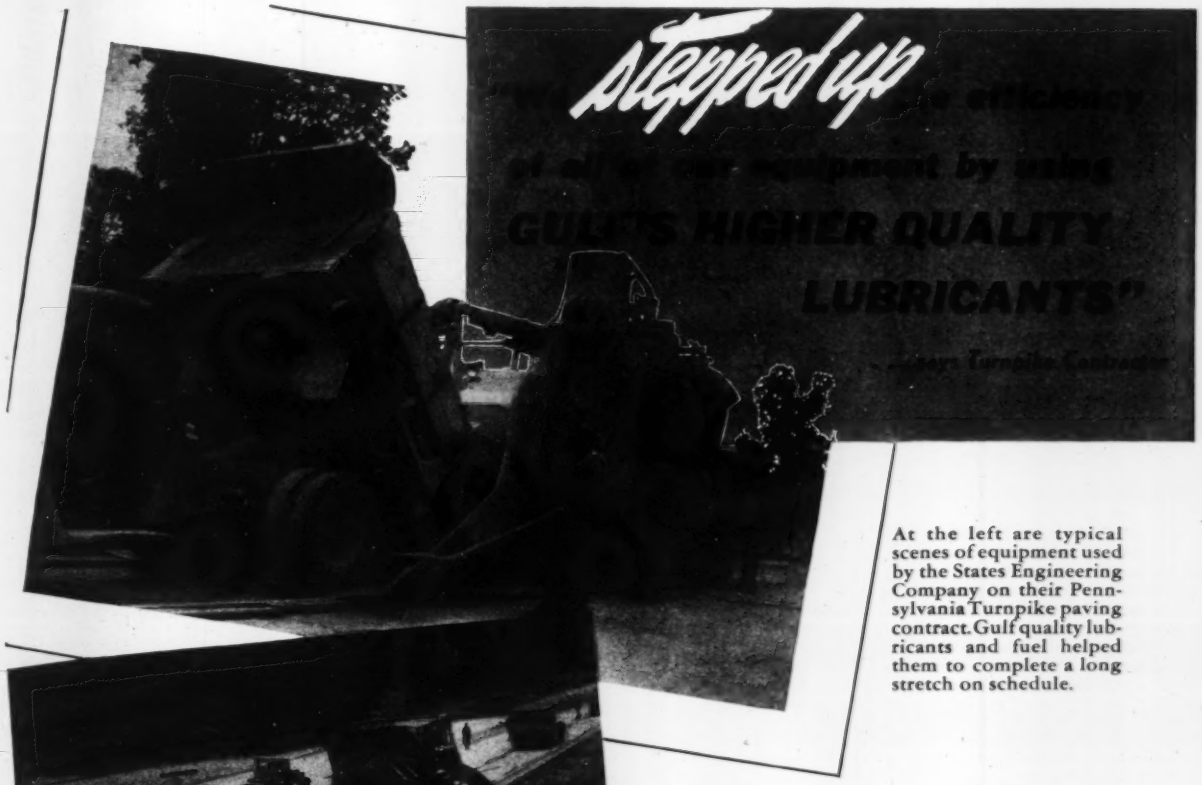
FLOW OF THE RIO GRANDE AND TRIBUTARY CONTRIBUTIONS—1939. Prepared by the International Boundary Commission, United States and Mexico. United States Section, 627 First National Bank Bldg., El Paso, Tex.

PHOTOELASTIC ANALYSIS OF TWO- AND THREE-DIMENSIONAL STRESS SYSTEMS—Bulletin No. 106. Published by The Engineering Experiment Station, The Ohio State Univ., Columbus, Ohio. Price 40c.

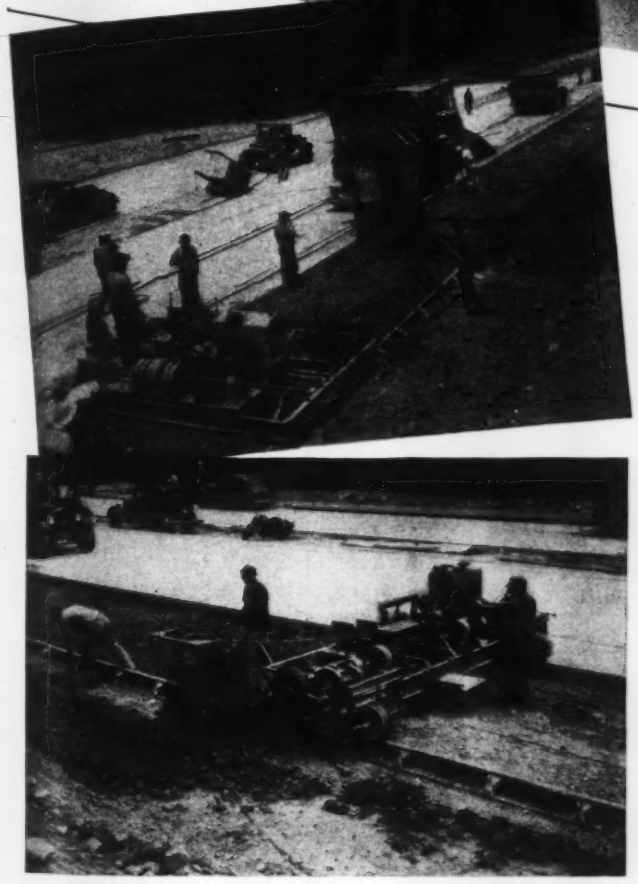
RESULTS OF COMPARATIVE PERFORMANCE TESTS AT PRINCETON, N. J., on Beams Reinforced with Twin Twisted and Stretched Reinforcing Steel vs. Standard Reinforcing Steel—By Elwyn E. Seelye, 101 Park Ave., New York.

A STUDY OF LUMBER AND PLYWOOD JOINTS WITH METAL SPLIT-RING CONNECTORS—Bulletin No. 53. Published by the Engineering Experiment Station, Pennsylvania State College, State College, Pa. Price $1.

(*Continued on p. 82*)



At the left are typical scenes of equipment used by the States Engineering Company on their Pennsylvania Turnpike paving contract. Gulf quality lubricants and fuel helped them to complete a long stretch on schedule.



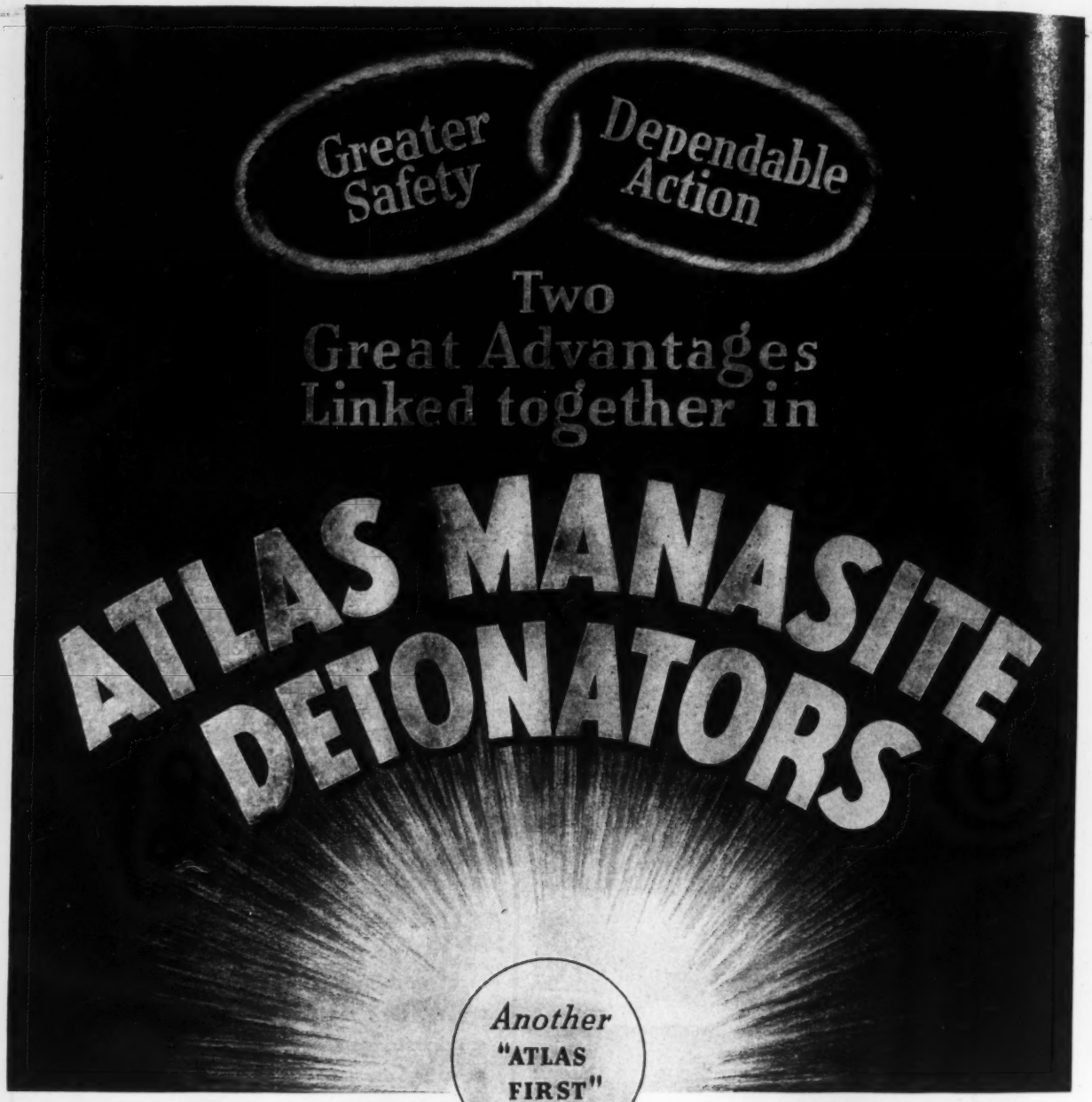
States Engineering Company of Harrisburg, Pa. and Rapid City, South Dakota, paved approximately 5 miles of the Pennsylvania Turnpike in Franklin and Cumberland Counties. They operate the most modern paving equipment available. To keep this costly machinery in top-notch operating condition, they standardized on Gulf quality lubricants and fuels.

"We followed the Gulf Engineer's Recommendations and avoided machinery breakdowns"

"WE standardized on Gulf quality lubricants to get the utmost efficiency from our equipment on this job," says this contractor. "Working against time meant that we had to be sure of continuous operation. Gulf's higher quality lubricants and the practical assistance of the Gulf lubrication engineer gave us that assurance."

Call in the Gulf engineer before you start your next big job—he can help you get extra hours of trouble-free operation, at no additional cost. Gulf's higher quality lubricants are quickly available, no matter where your job is located, from 1,100 strategically located warehouses in 30 states, from Maine to New Mexico. 'Phone or write your nearest Gulf office today!

GULF



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R. L. LAWRENCE, Jr.

(Superintendent and Chief Engineer, Water Works Department, Nashville, Tennessee)

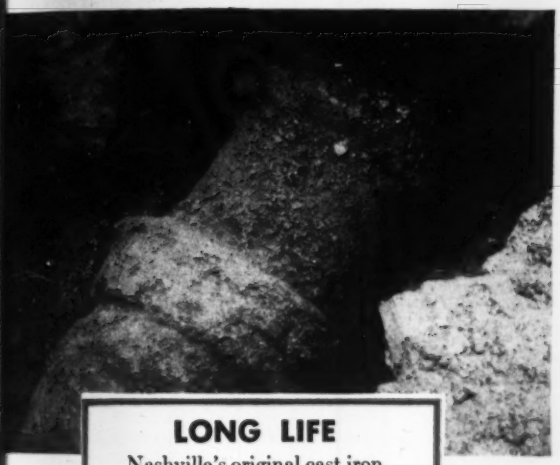
writes:

"Although we have cast iron pipe in our system over 100 years of age, the only photographs available are of 24-inch pipe laid 59 years ago in the congested business section of the City. Several hundred feet of this line, which traversed the site of our new City-County Building, was recently removed and, being in excellent condition, will ultimately be placed back in service elsewhere. The maintenance cost per mile of this old line has been approximately $5.44 per year."

R. L. Lawrence Jr.

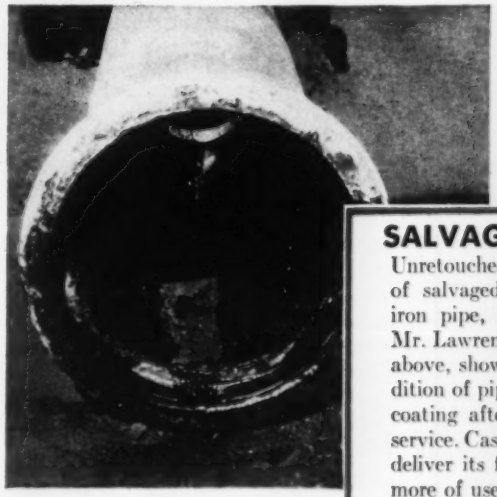
LOW MAINTENANCE

is an economy feature of cast iron pipe that is proved by water works records the nation over. An impartial survey among 195 water works superintendents shows that the maintenance cost of cast iron pipe is far below that of any other pipe material which has been in use long enough for the recording of conclusive data.



LONG LIFE

Nashville's original cast iron water main, still in service, was laid one year after New York's 109 year-old cast iron main, shown in the accompanying unretouched photograph. Many millions in tax dollars have been saved by the long life, low maintenance cost and salvage value of cast iron pipe.



SALVAGE VALUE

Unretouched photograph of salvaged 24-inch cast iron pipe, referred to in Mr. Lawrence's statement above, showing good condition of pipe and interior coating after 59 years of service. Cast iron pipe will deliver its full century or more of useful life, in one location or another, as changed conditions may require.

Water mains represent about one-third of this country's 5-billion-dollar investment in public water supply systems. More than 98% of these mains are cast iron pipe with a *known* useful life at least double the *estimated* life of other water main materials. Because the tax-saving, through avoiding replacements, is enormous, cast iron pipe is known as Public Tax Saver No. 1.

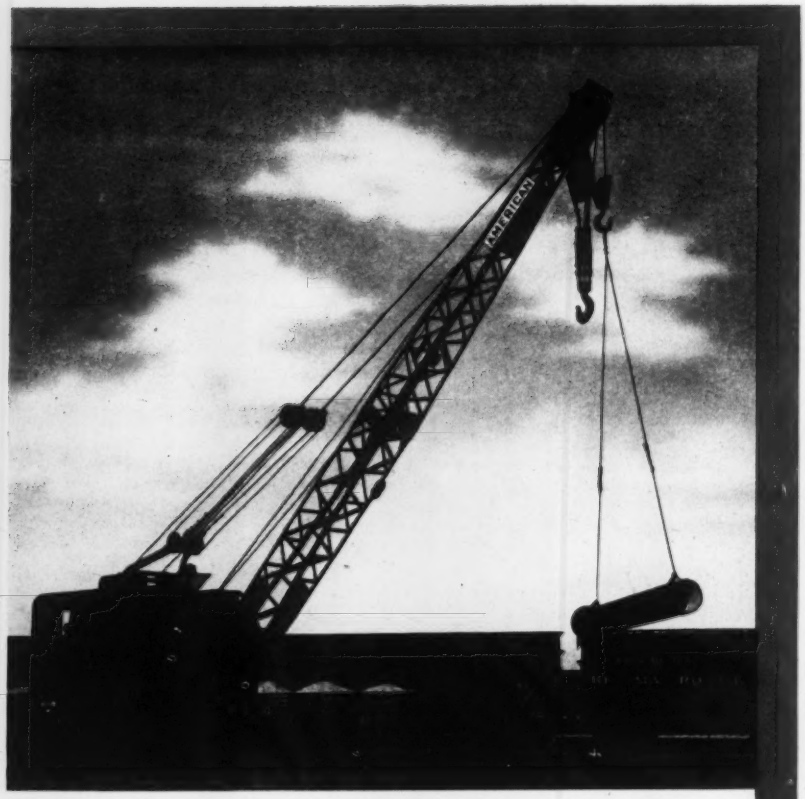
CAST Q IRON

Look for the "Q-Check" Registered trade mark. Cast iron pipe is made in diameters from 1¼ to 84 inches.

CAST IRON PIPE RESEARCH ASSOCIATION, THOMAS F. WOLFE, RESEARCH ENGINEER, 1013 PEOPLES GAS BUILDING, CHICAGO, ILLINOIS

CAST IRON PIPE

PUBLIC TAX SAVER NO. 1



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Model 830 — 30 ton Model 508 — 50 ton

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PERFECT GRIP . . . DROP FORGED STEEL . . . NOT GALVANIZED

DEALERS WITH STOCK IN ALL PRINCIPAL CITIES

(Continued from p. 78)

THE MISSISSIPPI RIVER—A S[illegible] Historic Description of the D[illegible]ment of Flood Control and Naviga[illegible] Prepared in the Office of the [illegible]dent, Mississippi River Commissio[illegible] Vicksburg, Miss.

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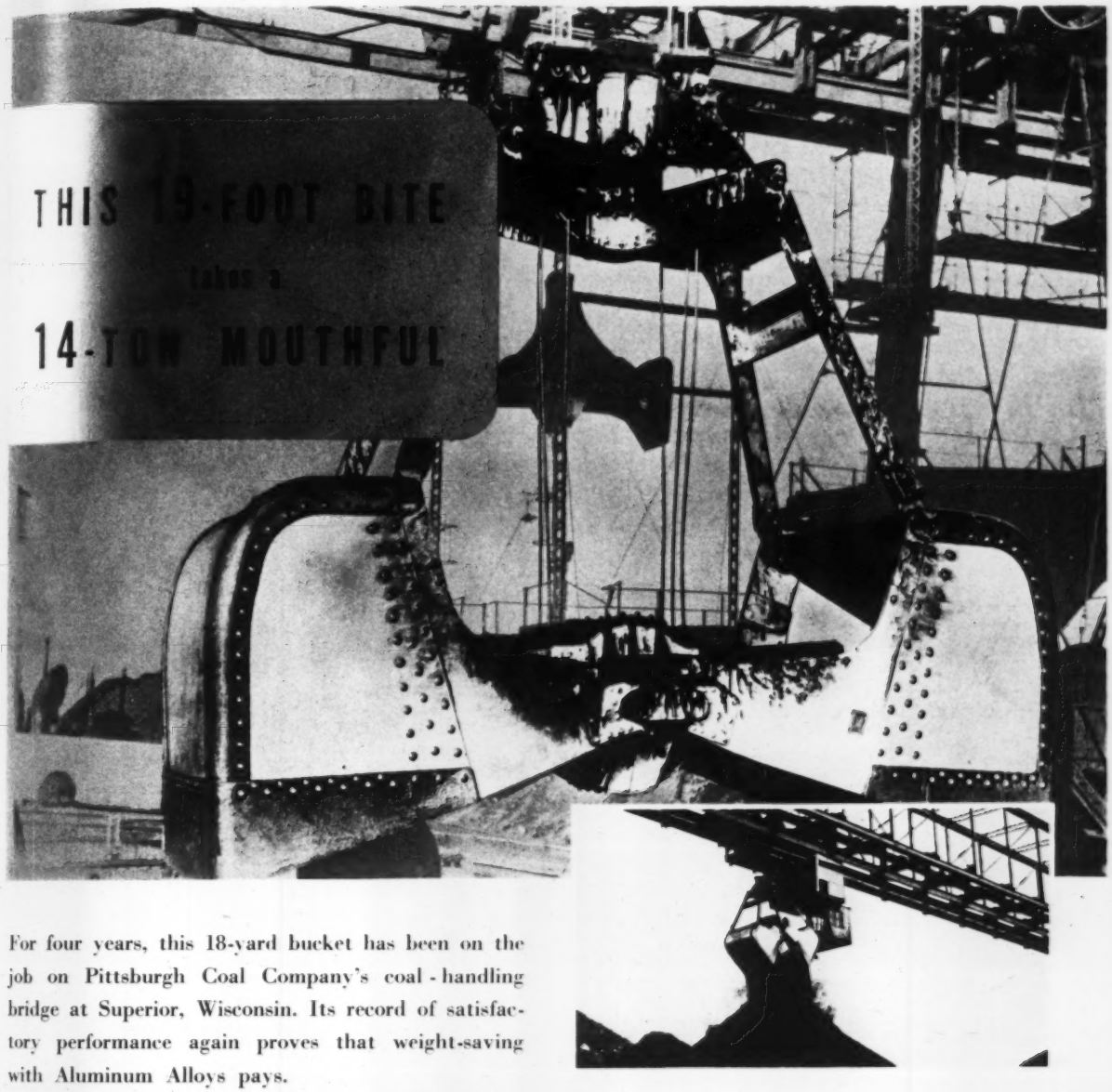
THE MILWAUKEE RIVER BASIN—Published by the Wisconsin State Planning Board, Madison, Wis.



For four years, this 18-yard bucket has been on the job on Pittsburgh Coal Company's coal-handling bridge at Superior, Wisconsin. Its record of satisfactory performance again proves that weight-saving with Aluminum Alloys pays.

4,225 pounds of Aluminum went into the building of this clamshell bucket; for blade arms and braces, side and end body plates, and guard plates. The complete bucket weighs 22,500 pounds, which is less than its rated 25,000-pound capacity. Yet it digs in and fills completely, demonstrating that excessive weight is not necessary for the job.

Similar weight reductions, using Alcoa Aluminum Alloys, were made on the crane trolley at the time this large bucket was installed. Operations were speeded up so much that a second bridge could be abandoned. The combination resulted in a reduction of power costs, maintenance, and labor.

ALUMINUM COMPANY OF AMERICA, 2108 Gulf Building, Pittsburgh, Pennsylvania.

ALCOA ALUMINUM

CONTRACT UNIT PRICES

What contractors are bidding on different kinds of construction work

CANTILEVER BRIDGE WASHINGTON

OWNER: Washington State Highway Department, Olympia.

PROJECT: Superstructure of 3-span, steel, cantilever bridge 756 ft. long over Spokane River, Primary S.H. 22, near Lincoln, in Lincoln and Stevens Counties, Wash. Anchor spans each 162-ft. long; main span 432-ft. long with 162-ft. suspended member. Concrete roadway 24-ft. wide, sidewalk (1) 3½-ft. wide. Steel trusses 60-ft. deep at pier and 28-ft. deep at centerline of main span. Clearance 45-ft. at maximum elevation of water in Grand Coulee Dam Reservoir.

CONDITIONS: Contractor to furnish all materials and complete project in 270 days. Highway facilities available at site of work. Nearest railroad at Davenport, 25 miles away. Contractor intends to truck steel from Seattle, 350 miles. Wage rates specified are: skilled labor, $1.25 per hr.; semi-skilled, 75c.; and common, 65c.

BIDS: Four bids were received October 22, 1940, ranging from the contract low of $199,187 to $223,445.

LIST OF BIDDERS:

1. C. & F. Teaming & Trucking Co., Butte, Mont. $199,187
2. MacRae Brothers, Seattle, Wash. 204,565
3. Angeles Gravel & Supply Co., Port Angeles, Wash. 212,519

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Class A concrete	465 c. y.	$30.00	$30.00	$27.00
2. Steel reinf. bars	126,000 lb.	.06	.05	.06
3. Structural carbon steel	1,543,000 lb.	.082	.0865	.09
4. Structural silicon steel	408,000 lb.	.09	.10	.10
5. Cast steel	41,000 lb.	.20	.14	.18
6. Cast steel drains	22 ea.	25.00	25.00	30.00
7. Steel sidewalk railings	750 l. f.	4.00	3.00	4.00
8. Copper seals	155 lb.	1.00	1.00	1.25
9. Lighting system	L.S.	1,600.00	1,350.00	1,500.00

HIGHWAYS, VERMONT

OWNER: Vermont State Highway Department, H. E. Sargent, chief engineer, Montpelier, Vt.; O. A. Webster, engineer.

PROJECT: Improving 0.681 miles on Route 14 in Sharon Village, Vt. Involves grading, drainage and sub-base with double tack coat of refined tar. Graded section 33-ft. minimum shoulder to shoulder. Width of tack coat 22-ft.

CONDITIONS: Contractor to furnish all materials and complete work by July 1, 1941. Rail and highway transportation facilities available. Site of work makes construction difficult because of steep bank to river on one side and cemetery requiring large wall on other. Wage rates specified are: skilled labor, 60c. per hr.; semi-skilled, 40c., common, 33⅓c.

BIDS: Eighteen bids were opened September 27, 1940, ranging from the contract low of $49,208 to $69,779.

LIST OF BIDDERS:

1. S. B. Manning, White River Junction, Vt. (contract) $49,208
2. Lane Construction Corp., Meriden, Conn. 49,289
3. Lambert & George, Montpelier, Vt. 49,641
4. Troy Paving Co., Inc., Hudson Falls, N. Y. 52,548
5. John Iafolla Constr. Co., Dedham, Mass. 54,323
6. D. W. Overocker, Inc., Burlington, Vt. 57,408
7. Lovelace & Schumacher, Inc., W. Springfield, Mass. 57,854
8. Littleton Constr. Co., Littleton, N. H. 57,862
9. Prairie Bros., Alburg, Vt. 57,990
10. Charter Oak Constr. Co., Inc., Hartford, Conn. 61,609
11. C. L. Hale Constr. Co., Manchester, Conn. 61,663
12. Kelleher Corp., Turner Falls, Mass. 61,703
13. Bristol Constr. Co., Inc., Bristol, N. H. 62,599
14. Rome Constr. Co., Inc., Holden, Mass. 64,182
15. Derrick DeGroot, Jericho, Vt. 64,358
16. Frank J. Fabian, Poultney, Vt. 65,700
17. Patterson & Rossi, Inc., Torrington, Conn. 69,777
18. Peter Salvucci, Waltham, Mass. 69,779

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Solid rock excavation	100 c. y.	$1.00	$2.00	$0.19
2. Common excav., incl. borrow	117,600 c. y.	.18	.20	.19
3. Trench excavation, earth	390 c. y.	1.00	.75	.75
4. Trench excavation, rock	15 c. y.	1.00	3.50	.75
5. Clearing and grubbing	3.5 ac.	60.00	100.00	100.00
6. Channel excavation	400 c. y.	.75	.50	.50
7. Structure excavation	1,015 c. y.	1.00	1.25	.50
8. Patrol maintenance	4 da.	25.00	15.00	10.00
9. Sub-base gravel	8,100 c. y.	.35	.35	.35
10. Double tack coat of refined tar	7,370 gal.	.15	.13	.13
11. Concrete Class A	252 c. y.	16.50	17.00	18.00
12. Reinforcing steel	44,670 lb.	.045	.04	.05
13. Cement rubble masonry	810 c. y.	10.00	7.00	9.00
14. 12-in. rein. conc. pipe	240 l. f.	1.15	1.25	1.45
15. 18-in. rein. conc. pipe	128 l. f.	2.10	2.25	2.25
16. 12-in. corr. galv. met. pipe, asph. coated	48 l. f.	1.50	1.25	1.65
17. 18-in. corr. galv. met. pipe, asph. coated	128 l. f.	1.95	2.00	2.15
18. Relaying pipe culverts	50 l. f.	.10	.25	.50
19. Stone fill	3,300 c. y.	1.00	1.25	1.00
20. Std. underdrain with 6-in. perf. corr. galv. metal pipe, asph. coat.	500 l. f.	.75	.75	.75
21. Wood guard rail	2,944 l. f.	.65	.50	.60
22. Guide posts	39 ea.	1.50	1.75	2.00
23. Erection of state or F. A. markers	1 ea.	5.00	10.00	15.00
24. Boundary markers	20 ea.	5.00	3.50	3.00
25. Drop inlet grates	2 ea.	10.00	10.00	15.00
26. One course cem. conc. sidewalk	75 s. y.	1.80	2.25	2.00
27. Creosoted timber curb	308 l. f.	.35	.25	.30
28. Finish grad., fertiliz. and seeding	300 s. y.	.15	.03	.05
29. Finish grad., fertiliz. and sodding	1,000 s. y.	.30	.10	.30
30. Comm'l fertilizer for grasses	175 lb.	.10	.05	.30
31. Grass seed	15 lb.	.20	.40	.30
32. Grass sod	1,000 s. y.	.30	.25	.50
33. Topsoil	120 c. y.	1.00	.75	1.00
34. Plant trees, acer saccharum, 8 to 10-ft.	1 ea.	25.00	10.00	10.00
35. Cut and remove trees	7 ea.	5.00	5.00	20.00

HIGHWAYS, TENNESSEE

OWNER: Tennessee Department of Highways and Public Works, Nashville, Tenn.; R. J. Love, division engineer.

PROJECT: Construction of 8.132 miles of plain cement concrete roadway, State Highway 11, between Ridgetop and Springfield, Tenn.

CONDITIONS: Contractor to furnish all materials and complete work in 100 working days. Rail and highway transportation facilities available. Ten mile haul from Cumberland River required for sand and gravel for project. Minimum wage rates specified are: skilled labor, 75c. per hr.; semi-skilled, 45c.; and common labor, 30c. per hr.

BIDS: Ten bids were opened October 4, 1940, ranging from the contract low of $163,832 to $205,117.

LIST OF BIDDERS:

1. H. E. Wolfe Constr. Co., Inc., St. Augustine, Fla. $163,832
2. F. C. Gorrell & Sons, Russellville, Ky. 165,472
3. Weymouth Constr. Co., Marks, Miss. 170,431
4. J. B. Michael & Co., Memphis, Tenn. 170,836
5. Bowyer & Johnson, Jackson, Tenn. 174,512
6. Pittman Constr. Co., Atlanta, Ga. 175,827
7. Stacy Brothers Co., Johnson City, Tenn. 179,981
8. Foster & Creighton Co., Nashville, Tenn. 186,868
9. Ziegler Constr. Co., Nashville, Tenn. 202,520
10. Municipal Paving & Constr. Co., Nashville, Tenn. 205,117

Item	Quan.	Unit Prices (1)	(2)	(3)
1. Road and drainage excav. (uncl.)	19,516 c. y.	$0.15	$0.15	$0.30
2. Culvert excav., unclassified	83 c. y.	.75	1.00	1.00
3. Crushed stone (tr. bd. rdwy.)	1,237 c. y.	2.25	1.85	2.50
4. Cem. conc. pavement, plain (8-6-8-6-8)	104,968 s. y.	1.50	1.52	1.53
5. Class B concrete	23.3 c. y.	20.00	20.00	25.00
6. Reinforcing steel bars	760 lb.	.05	.05	.10
7. 15-in. corr. met. pipe (side-drain)	94 l. f.	1.10	1.25	1.50